MAINE DEPARTMENT OF TRANSPORTATION
BRIDGE PROGRAM
GEOTECHNICAL SECTION
AUGUSTA, MAINE

GEOTECHNICAL DESIGN REPORT

For the Replacement of:

CANAL STREET BRIDGE
OVER WILSON STREAM
WILTON, MAINE

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GEOTECHNICAL DESIGN SUMMARY

The purpose of this report is to present subsurface information and make geotechnical recommendations for the replacement of the Canal Street Bridge over Wilson Stream in Wilton, Maine. The proposed bridge replacement will consist of an approximately 60 foot single-span, single-lane structure with a sidewalk. The structure will be advertised as a Detail-Build component on the project plans. The bridge will be supported on H-pile supported integral abutments. The following design recommendations are discussed in detail in the attached report:

Integral Abutment H-Piles – The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. However, it should be noted that the borings encountered frequent cobbles and boulders within the fill behind the existing abutment locations. Construction problems should therefore be anticipated with respect to piles being out of position or refusing on obstructions above the specified tip elevation. The use of short pile supported integral abutments is under consideration by the MaineDOT Bridge Program. Initial results indicate that although fixity is not achieved for piles less than 13 feet long, the piles do not experience stresses larger than those seen by longer integral abutment piles. Short piles supporting integral abutments should be designed in accordance with AASHTO LRFD and the pile tips analyzed for frictional resistance per the design example found in Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" Chapter 5 and Appendix B. The Contractor is required to perform a wave equation analysis and dynamic pile analysis. The nominal pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the maximum factored axial pile load divided by a resistance factor of 0.65. The maximum factored pile load should be shown on the plans. The piles should be oriented for weak axis bending. Driven piles should be fitted with a Rock Injector HP-80500 Pile Point, manufactured by Associated Pile and Fitting, LLC to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption.

Integral Stub Abutments – Integral stub abutments shall be designed for all relevant strength, service and extreme limit states and load combinations. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforce concrete failure. Strength limit state design shall also consider change in foundation conditions and pile group resistance after scour due to the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination. Extreme limit state design checks for abutments supported on piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads.

Cast-in-place integral abutments sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure state. The Coulomb passive earth pressure coefficient, K_p, of 6.89 is recommended. Developing full passive requires displacements of

the abutment on the order of 2 to 5 percent of the abutment height. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction, or designing using a reduced Coulomb passive earth pressure coefficient, but not less than the Rankine passive earth pressure case using a Rankine passive earth pressure coefficient, K_p, of 3.25. A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, $\gamma_{EH} = 1.50$.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted.

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. The approach slab should be positively connected to the abutment.

Scour and Riprap – If using integral abutments at the site, pile lengths at Abutment No. 1 will be short and, therefore, scour protection will be critical. The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the unfactored extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour due to the design flood for scour.

For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. Stone riprap shall conform to item number 703.26 of the MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a Class 1 nonwoven erosion control geotextile and a 1 foot thick layer of bedding material.

Settlement - The roadway profile will be raised approximately 1.5 ft at the south abutment. Settlements due to the addition of this fill have been calculated to be less than ½ inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible.

Frost Protection - Any foundation placed on granular soils should be founded a minimum of 7.5 feet below finished exterior grade for frost protection. Pile supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection.

Seismic Design Considerations – Seismic analysis is not required for single span bridges regardless of seismic zone. However, superstructure connections and minimum support lengths should be designed in accordance with LRFD requirements

Construction Considerations –Frequent boulders and cobbles were encountered within the existing fill at the site. There is potential for these obstructions to impact the pile driving and/or installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Clearing obstructions shall be specified as incidental to related pay items. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The removal of the existing structure may require the replacement of excavated soils with compacted granular fill prior to pile driving.

In some locations the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The Contractor should control groundwater, surface water infiltration and soil erosion during construction.

1.0 INTRODUCTION

The purpose of this Geotechnical Design Report is to present geotechnical recommendations for the replacement of the Canal Street Bridge over Wilson Stream in Wilton, Maine. A subsurface investigation at the site has been completed. The purpose of the investigation was to explore subsurface conditions at the site in order to develop geotechnical recommendations for the bridge replacement. This report presents the soils information obtained at the site, geotechnical design recommendations, and foundation recommendations.

The existing Canal Street Bridge carries Canal Street over Wilson Stream at Wilson Pond outlet and was constructed in 1955. The bridge consists of a single-span steel riveted thru girder system with a corrugated steel deck founded on mass concrete abutments on soil. The structure has a span length of approximately 51 feet on a 15 degree skew. The 2009 Maine Department of Transportation (MaineDOT) maintenance inspection reports indicate that the bridge deck is in poor condition (rating of 4), the superstructure is in critical condition (rating of 2) and the substructures are in satisfactory condition (rating of 6). The Bridge Sufficiency Rating is 17.8. The structure has a scour critical rating of "U - unknown scour" meaning that the foundations have not been evaluated for scour. Inspection records note that the bridge is in overall critical condition with extensive deterioration of the floor beam system. Large holes and heavy rusting are present in the girders and deck system. The bridge has a load rating of 3 tons. The existing bridge substructures are generally sound.

The MaineDOT Bridge Program is currently proposing a replacement structure consisting of a single-span, single-lane structure with a sidewalk founded on H-pile supported integral abutments. The bridge superstructure will be a detail-build option in the final construction contract. Due to the short pile length at Abutment No. 1, this abutment type is considered experimental and is proposed based on the results to date of MaineDOT's short-pile integral abutment study. Constructability issues associated with driving the piles through cobbles and boulders should be anticipated. The overall length of the proposed replacement structure will be increased from the existing 51 feet to approximately 60 feet. The proposed structure will have a skew of approximately 20 degrees. The roadway profile will be raised approximately 1.5 ft at the south abutment. The road and bridge will be closed during construction.

2.0 GEOLOGIC SETTING

The Canal Street Bridge in Wilton carries Canal Street over Wilson Stream at the Wilson Pond outlet as shown on Sheet 1 - Location Map found at the end of this report.

According to the Surficial Geologic Map of Maine published by the Maine Geological Survey (1985) the surficial soils in the vicinity of the site consist of till deposits. Soils in the site area are generally comprised of a heterogeneous mixture of sand silt clay and stones. The unit is generally a blanket deposit that conforms to the underlying bedrock topography. These soils were generally deposited by glacial ice.

According to the Bedrock Geologic Map of Maine, published by the Maine Geological Survey (1985), the site is located at the interface of two formations. The bedrock to the

southeast of the site is described as interbedded pelite and sandstone and is the Anasagunticook Member of the Sangerville Formation. The bedrock to the northwest of the site is described as calcareous sandstone and interbedded sandstone and impure limestone and is a part of the Madrid Formation.

3.0 SUBSURFACE INVESTIGATION

Subsurface conditions at the site were explored by excavating two (2) test pits and drilling two (2) test borings. Test pit TP-WWS-101 and test boring BB-WWS-101 were conducted behind the south abutment. Test pit TP-WWS-102 and test boring BB-WWS-102 were conducted behind the north abutment. The purpose of the test pits was to determine the actual dimension of the existing abutments.

The exploration locations and an interpretive subsurface profile depicting the site stratigraphy are shown on Sheet 2 - Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The test pits were excavated on September 7 and 8, 2010 by the Maine DOT Jay Maintenance crew. The borings were drilled on October 4 and 5, 2010 by the MaineDOT drill crew. Details and sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented in the test pit and boring logs provided in Appendix A – Test Pit and Boring Logs and on Sheet 3 – Test Pit and Boring Logs found end of this report.

The test pits were excavated using a Case 580 excavator. No soil samples were taken from the test pits. Test pits were excavated to the groundwater table.

The borings were drilled using solid stem auger and driven cased wash boring drilling techniques. Soil samples were obtained where possible at 5-foot intervals using Standard Penetration Test (SPT) methods. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6 inch interval of penetration are recorded. The standard penetration resistance, N-value, is the sum of the blows for the second and third intervals. MaineDOT drill rig is equipped with an automatic hammer to drive the split spoon. The hammer was calibrated in March of 2010 and was found to deliver approximately 40 percent more energy during driving than the standard rope and cathead system. All N-values discussed in this report are corrected values computed by applying an average energy transfer factor of 0.84 to the raw field N-values. This hammer efficiency factor (0.84) and both the raw field N-value and the corrected N-value are shown on the boring logs. The bedrock was cored in the borings using an NQ-2” core barrel and the Rock Quality Designation (RQD) of the core was calculated.

The MaineDOT geotechnical team member selected the test pit and boring locations and drilling methods, designated type and depth of sampling techniques and identified field and laboratory testing requirements. A Northeast Transportation Technician Certification Program (NETTCP) Certified Subsurface Inspector or the geotechnical team member logged the subsurface conditions encountered. The test pits and borings were located in the field by use of a tape after completion of the exploration programs.

4.0 LABORATORY TESTING

Laboratory testing for samples obtained in the borings consisted of two (2) standard grain size analyses with water content and five (5) grain size analyses with hydrometer and water content. The results of these laboratory tests are provided in Appendix B - Laboratory Data at the end of this report. Moisture content information and other soil test results are included on the Boring Logs in Appendix A and on Sheet 3 – Test Pit and Boring Logs found at the end of this report.

5.0 SUBSURFACE CONDITIONS ENCOUNTERED IN TEST PITS

The test pits were excavated to determine the existing conditions and dimensions of the abutments. No soils samples were take in the test pits. Soils encountered in both of the test pits were brown, moist, fine to coarse sand with frequent cobbles and boulders. Layers of old pavement were observed in test pit TP-WWS-101 at a depth of approximately 4.0 feet. Both test pits were terminated at a depth of approximately 8.5 feet which coincided with the groundwater surface. Test pits cannot be effectively excavated below the groundwater table.

Abutment No. 1 (Main Street side) was found to have a height of approximately 6.0 feet from the existing roadway surface. The abutment has no footing and is sitting on sand and boulder fill. The batter of the abutment wall on the backside was found to be 3” in 12” (1H:4V). This abutment is not currently embedded for frost protection.

The full height of Abutment No. 2 (boat ramp side) was not observed in the test pit. Measurements of the abutment on the water side indicate that the abutment has a height of approximately 11.0 feet. A short toe was observed in the water at the front of the abutment having a shelf length of approximately 1.1 feet. The depth of the toe was not determined. It is not known if the toe continues to a heel at the back of the abutment. The batter of the abutment wall on the backside was found to be 3” in 12” (1H:4V).

6.0 SUBSURFACE CONDITIONS ENCOUNTERED IN BORINGS

Subsurface conditions encountered at the borings generally consisted of sand fill, underlain by a layer of native sandy silt and silt, underlain silty sand, all underlain by bedrock. An interpretive subsurface profile depicting the site stratigraphy is shown on Sheet 2 – Boring Location Plan and Interpretive Subsurface Profile found at the end of this report. The following paragraphs discuss the subsurface conditions encountered in the borings in detail:

6.1 Sand Fill

A layer of sand fill, with frequent cobbles and boulders was encountered beneath the pavement in both of the borings and the test pits. The thickness of the fill layer ranged from approximately 8.5 feet in boring BB-WWS-101 to approximately 9.0 in boring BB-WWS-102. The soil generally consisted of:

- Brown, moist, fine to coarse sand with some silt, trace gravel and frequent cobbles and boulders,
- Brown, damp, gravelly, fine to coarse sand with little silt and frequent cobbles and boulders, and
- Brown, wet, fine to coarse sand with some gravel, some silt, trace clay and frequent cobbles and boulders.

Corrected SPT N-values in the fill unit ranged from 4 to 31 blows per foot (bpf) indicating that the fill is loose to dense in consistency. Water contents from three (3) samples obtained within the fill layer range from approximately 4% to 12%. Three (3) grain size analyses conducted on samples of the fill indicate that the soil is classified as an A-2-4 or A-1-a by the AASHTO Classification System and a SM or SC-SM by the Unified Soil Classification System.

6.2 Native Sandy Silt and Silt

A layer of native sandy silt and silt was encountered beneath the fill. The thickness of the layer ranged from approximately 9.9 feet in boring BB-WWS-101 to approximately 15.0 feet thick boring BB-WWS-102. The layer generally consisted of:

- Olive, wet, fine to coarse sandy silt with little clay and little gravel,
- Brown, wet, fine to coarse sandy silt with little clay and trace gravel, and
- Grey, wet, fine to coarse sandy silt with trace to little clay and trace to little gravel.

Corrected SPT N-values in the layer ranged from 6 to 83 bpf indicating that the soil is medium stiff to hard in consistency. Water contents from three (3) samples obtained within the layer range from approximately 10% to 16%. Three (3) grain size analyses conducted on samples from the layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a CL-ML or SC-SM by the Unified Soil Classification System.

6.3 Silty Sand

A layer of silty sand was encountered beneath the sandy silt and silt in boring BB-WWS-102. The thickness of the layer was approximately 2.2 feet. The silty sand generally consisted of grey, wet, silty fine to coarse sand with little clay and trace gravel.

One corrected SPT N-value in the layer was 81 bpf indicating that the soil is very dense in consistency. Water contents from a sample obtained within the layer range was approximately 9%. One (1) grain size analysis conducted on a sample from the layer indicate that the soil is classified as an A-4 by the AASHTO Classification System and a CL-ML by the Unified Soil Classification System.

6.4 Bedrock

Bedrock was encountered and cored in both of the borings. The Table 5-1 summarizes the depths to bedrock and corresponding elevations of the top of bedrock:

Boring Number	Depth to Bedrock	Bedrock Elevation	RQD
BB- WWS -101	18.4 feet	559.9 feet	52 – 68%
BB- WWS -102	26.2 feet	550.8 feet	11 – 60%

Table 6-1 - Summary of Bedrock Depths, Elevations and RQD

The bedrock is identified as grey, banded, augen gneiss with joints along the foliation dipping 10 to 25 degrees. The augen are typically rectangular and may be amphibole with inclusions of garnet. Iron staining is present especially at the joints and in the fractured areas. The rock quality designation (RQD) of the bedrock was determined to range from 11 to 68 percent indicating a rock mass quality of very poor to fair.

6.5 Groundwater

Groundwater was observed at a depth of approximately 9.0 feet below the existing ground surface in both of the borings and a depth of approximately 8.5 feet below the existing ground surface in both of the test pits. The water levels measured upon completion of drilling are indicated on the boring logs found in Appendix A. Note that water was introduced into the boreholes during the drilling operations. It is likely that the water levels indicated on the boring logs do not represent stabilized groundwater conditions. Additionally, groundwater levels are expected to fluctuate seasonally depending upon the local precipitation magnitudes.

7.0 FOUNDATION ALTERNATIVES

The following foundation alternatives were considered for the bridge replacement:

- Spread footings founded on soil,
- Spread footings founded on concrete seals on soil, and
- Driven pile supported integral abutments

After consideration of all of the alternatives, integral abutments located behind the existing abutments were selected because they require minimal future maintenance. The presence of cobbles and boulders in the fill materials directly behind both of the abutments may impede driving piles within acceptable tolerances. Construction delays in order to excavate obstructions or preauger or drive spud piles should be anticipated. A note on the Plans should alert the Contractor to this issue and specify that clearing obstructions will be incidental to related pile pay items. Pile order lengths should include additional pile to replace pile lengths damaged during driving. The presence of relatively shallow bedrock at the Abutment No. 1 indicates that integral abutment piles would typically be socketed in

bedrock to achieve fixity. Preliminary results of a MaineDOT short-pile integral abutment study show that fixity may not be necessary.

8.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections will discuss geotechnical design recommendations for stub abutments founded on a single row of integral H-piles driven to bedrock which has been identified as the optimal substructure for the site. The use of short pile supported integral abutments is under consideration by the MaineDOT Bridge Program. Initial results indicate that although fixity is not achieved for piles less than 13 feet long, short integral piles do not experience stresses larger than those seen by longer integral abutment piles. The current study[1] indicates that the use of short pile supported integral abutments for bridges with spans not exceeding 115 feet is applicable.

8.1 Integral Abutment H-Piles

The use of stub abutments founded on a single row of driven integral H-piles is a viable foundation system for use at the site. The piles should be end bearing, driven to the required resistance on or within the bedrock. Piles may be HP 12x53, HP 12x74, HP 14x73, HP 14x89, or HP 14x117 depending on the factored design axial loads. Piles should be 50 ksi, Grade A572 steel H-piles. The piles should be oriented for weak axis bending. Piles should be fitted with Rock Injector HP-80500 Pile Point, manufactured by Associated Pile and Fitting, LLC to protect the tips, improve penetration and improve friction at the pile tip to support a pinned pile tip assumption. Special Provision 501 Foundation Piles – Rock Injector Pile Tip is provided in Appendix D – Special Provisions found at the end of this report.

Pile lengths at the proposed abutments may be estimated based on Table 8-1 below:

Location	Estimated Pile Cap Bottom Elevation	Approximate Depth to Bedrock From Ground Surface	Approximate Top of Rock Elevation	Estimated Pile Free Length
Abutment #1 BB-WWS-101	572.1 feet	18.4 feet	559.9 feet	12 feet
Abutment #2 BB-WWS-102	570.2 feet	26.2 feet	550.8 feet	19 feet

Table 8-1 – Estimated Pile Lengths for Plumb H-Piles

These pile lengths do not take into account the length of pile embedded in the pile cap, the additional five (5) feet of pile required for dynamic testing instrumentation or any additional

[1] MaineDOT Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1"

pile length needed to accommodate damaged pile lengths and the Contractor's leads and driving equipment.

The H-piles shall be designed for the strength limit state considering the structural resistance of the piles, the geotechnical resistance of the pile and loss of the lateral support due to scour at the design flood event. The structural resistance check should include checking axial, lateral, and flexural resistance. Resistance factors for use in the design of piles at the strength limit state are discussed in Section 8.1.1 below. Short piles (less than 12 feet) should be designed in accordance with AASHTO LRFD Bridge Design Specifications 5th Edition (LRFD) and pile tip frictional resistance checked with the design example found in Technical Report ME-01-7, June 2005, "Behavior of Pile Supported Integral Abutments at Bridge Sites with Shallow Bedrock - Phase 1" Chapter 5 and Appendix B. The H-piles shall also be checked for fixity and combined axial and flexure using LPile® software.

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and displacements considering changes in foundation conditions due to scour at the design flood event. Extreme limit state design shall check that the nominal pile resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0. The design and check floods for scour are defined in LRFD Articles 2.6.4.4.2 and 3.7.5.

Since the abutment piles will be subjected to lateral loading, piles should be analyzed for axial loading and combined axial and lateral loading as defined in LRFD Article 6.15.2 and specified in LRFD Article 6.9.2.2. As the proposed piles for Abutment No. 1 will be short and will not achieve fixity, the resistance for these pile will be determined for structural compliance with interaction equation.

8.1.1 Strength Limit State Design

The nominal compressive resistance (P_n) in the strength limit state for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. It is the responsibility of the structural engineer to recalculate the nominal structural compressive resistance (P_n) based on "actual unbraced pile length (ℓ) and effective length factor (K)" or "on the actual elastic critical buckling resistance, P_e". Preliminary estimates of the factored structural axial compressive resistances of the five (5) proposed H-pile sections were calculated using a resistance factor, ϕ_c, of 0.50 (severe driving conditions) and an unbraced length (ℓ) of 48 inches and an effective length factor (K) of 2.0.

The nominal geotechnical compressive resistance in the strength limit state was calculated using Canadian Foundation Engineering Manual methods. The factored geotechnical compressive resistances of the four proposed H-pile sections were calculated using a resistance factor, φ_{stat}, of 0.45.

The drivability of the five (5) proposed H-pile sections was considered. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. As the piles will be driven to refusal on bedrock a drivability analysis to determine the resistance

that must be achieved was conducted. The resistance factor for a single pile in axial compression when a dynamic test is done, given in LRFD Table 10.5.5.2.3-1, is φ_{dyn}= 0.65.

For the strength limit state, the calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections are summarized in Table 8-2 below. Supporting calculations are included in Appendix C- Calculations found at the end of this report.

Pile Section	Strength Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* ϕ_c=0.50	Geotechnical Resistance φ_{stat}=0.45	Drivability Resistance φ_{dyn}=0.65	Governing Resistance
HP 12x53	357	251	295	251
HP 12x74	504	350	363	350
HP 14x73	506	312	354	312
HP 14x89	618	379	457	379
HP 14x117	816	497	634	497

* based on preliminary assumption of ℓ=48" and K=2.0

Table 8-2 - Factored Axial Resistances for Abutment Piles at the Strength Limit State

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial geotechnical resistance is less than the factored axial structural resistance and the factored axial drivability resistance. Therefore, it is recommended that the maximum factored axial pile load used in design for the strength limit state should not exceed the factored axial geotechnical resistance shown in the last column of Table 8-2 above.

Per LRFD Article 6.5.4.2, at the strength limit state, for H-piles in compression and bending, the axial resistance factor ϕ_c=0.7 and the flexural resistance factor ϕ_f =1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2). The combined axial compression and flexure should be evaluated in accordance with the applicable sections of LRFD Articles 6.9.2.2 and 6.15.2.

8.1.2 Service and Extreme Limit State Design

The design of the H-piles at the service limit state shall consider tolerable horizontal movement of the piles, overall stability of the pile group and displacements considering changes in foundation conditions due to scour at the design flood event. The extreme limit state design shall include a determination that there is adequate nominal foundation resistance remaining after scour due to the check flood to resist the unfactored extreme limit state load combination.

For the service and extreme limit states resistance factors, φ, of 1.0 are recommended for structural and geotechnical pile resistances. It is the responsibility of the structural engineer to recalculate P_n based on refined elastic critical buckling resistance (P_e) evaluations.

For the service and extreme limit states, the calculated factored axial compressive structural, geotechnical and drivability resistances of the five (5) proposed H-pile sections are summarized in Table 8-3 below. Supporting calculations are included in Appendix C-Calculations found at the end of this report.

Pile Section	Service and Extreme Limit State Factored Axial Pile Resistance (kips)			
	Structural Resistance* ϕ=1.0	Geotechnical Resistance φ=1.0	Drivability Resistance φ=1.0	Governing Resistance
HP 12x53	714	558	454	454
HP 12x74	1007	779	559	559
HP 14x73	1012	694	544	544
HP 14x89	1236	843	703	703
HP 14x117	1632	1104	976	976

*based on preliminary assumption of ℓ=48" and K=2.0

Table 8-3 - Factored Axial Resistances for Abutment Piles at the Service and Extreme Limit States

LRFD Article 10.7.3.2.3 states that the nominal resistance of piles driven to point bearing on hard rock where pile penetration into the rock formation is minimal is controlled by the structural limit state. However, the factored axial drivability resistance is less than the factored axial structural resistance and local experience supports the estimated factored resistance from the drivability analyses. Therefore, it is recommended that the maximum factored axial pile load used in design for the service and extreme limit states should not exceed the factored drivability resistance shown in the last column of Table 8-3 above.

8.1.3 Driven Pile Resistance and Pile Quality Control

The Contractor is required to perform a wave equation analysis of the proposed pile-hammer system and a dynamic pile test at each integral abutment. The first pile driven at each abutment should be dynamically tested to confirm capacity and verify the stopping criteria developed by the Contractor in the wave equation analysis. The ultimate pile resistance that must be achieved in the wave equation analysis and dynamic testing will be the factored axial pile load divided by a resistance factor of 0.65. The factored pile load should be shown on the plans.

Piles should be driven to an acceptable penetration resistance as determined by the Contractor based on the results of a wave equation analysis and as approved by the Resident. Driving stresses in the pile determined in the drivability analysis shall be less than 45 ksi in accordance with LRFD Article 10.7.8. A hammer should be selected which provides the required resistance when the penetration resistance for the final 3 to 6 inches is 3 to 15 blows

per inch. If an abrupt increase in driving resistance is encountered, the driving could be terminated when the penetration is less than 0.5-inch in 10 consecutive blows.

8.2 Integral Stub Abutment Design

Integral abutment sections shall be designed for all relevant strength, service and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. The design of pile supported abutments at the strength limit state shall consider pile group failure and structural reinforced concrete failure. Strength limit state design shall also consider changes in foundation conditions and pile group resistance after scour due to the design flood.

A resistance factor of ϕ= 1.0 shall be used to assess abutment design at the service limit state including: settlement, excessive horizontal movement and movement resulting from scour at the design flood. The overall global stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65.

Extreme limit state design checks for abutments supported on piles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal resistance remaining after scour due to the check flood can support the extreme limit state loads with a resistance factor of 1.0.

The Designer may assume Soil Type 4 (MaineDOT BDG Section 3.6.1) for backfill material soil properties. The backfill properties are as follows: ϕ = 32 degrees, γ = 125 pcf and a soil-concrete friction coefficient of 0.45. Cast-in-place integral abutment sections shall be designed to withstand a maximum applied lateral load equal to the passive earth pressure state. The Coulomb passive earth pressure coefficient, K_p, of 6.89 is recommended. Developing full passive requires displacements of the abutment on the order of 2 to 5 percent of the abutment height. If the calculated displacements are significantly less than that required to develop full passive pressure, the designer may consider using the Rankine passive earth pressure case, which assumes no wall friction, or designing using a reduced Coulomb passive earth pressure coefficient, but not less than the Rankine passive earth pressure case using a Rankine passive earth pressure coefficient, K_p, of 3.25. A load factor for passive earth pressure is not specified in LRFD. Use the maximum load factor for active earth pressure, γ_{EH} = 1.50.

Additional lateral earth pressure due to construction surcharge or live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height (h_{eq}) taken from Table 8-4 below:

Abutment Height	h_{eq}
5 feet	4.0 feet
10 feet	3.0 feet
≥20 feet	2.0 feet

Table 8-4 - Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

All abutment designs shall include a drainage system behind the abutments to intercept any groundwater. Drainage behind the structure shall be in accordance with Section 5.4.1.4 Drainage of the MaineDOT BDG. Geocomposite drainage board applied to the backsides of the abutments and wingwalls with weep holes will provide adequate drainage. The approach slab should be positively connected to the integral abutment.

Backfill within 10 feet of the abutments and wingwalls and side slope fill shall conform to Granular Borrow for Underwater Backfill - MaineDOT Specification 709.19. This gradation specifies 10 percent or less of the material passing the No. 200 sieve. This material is specified in order to reduce the amount of fines and to minimize frost action behind the structure.

Slopes in front of the pile supported integral abutments should be set back from the riverbank and should be constructed with riprap and erosion control geotextile. The slopes should not exceed 1.75H:1V.

8.3 Scour and Riprap

Grain size analyses were performed on soil samples taken at the approximate streambed elevation to generate grain size curves for determining parameters to be used in scour analyses. The samples were assumed to be similar in nature to the soils likely to be exposed to scour conditions. The following streambed grain size parameters can be used in scour analyses:

- Average diameter of particle at 50 percent passing, D_{50} = 0.06 mm
- Average diameter of particle at 95 percent passing, D_{95} = 8.8 mm
- Soil Classification AASHTO Soil Type A-4

The grain size curves are included in Appendix B- Laboratory Data found at the end of this report.

If using integral abutments at the site, pile lengths at Abutment No 1 will be short and, therefore, scour protection will be critical. The consequences of changes in foundation conditions resulting from the design and check floods for scour shall be considered at the strength and extreme limit states, respectively. Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to scour at the check flood event is no less than the unfactored extreme limit state loads. At the service limit state, the design shall limit movements and overall stability considering scour at the design load.

For scour protection and protection of pile groups, the bridge approach slopes and slopes at abutments should be armored with 3 feet of riprap. Refer to MaineDOT BDG Section 2.3.11 for information regarding scour design.

Riprap conforming to Special Provisions 610 and 703 shall be placed at the toes of arch footing and wingwalls. Special Provisions 610 and 703 are provided in Appendix D – Special Provisions found at the end of this report. Stone riprap shall conform to item number 703.26 of the MaineDOT Special Provision 703 and shall be placed at a maximum slope of 1.75H:1V. The toe of the riprap section shall be constructed 1 foot below the streambed elevation. The riprap section shall be underlain by a 1 foot thick layer of bedding material conforming to item number 703.19 of the Standard Specification and Class "1" Erosion Control Geotextile per Standard Details 610(02) through 610(04). Riprap shall be 3 feet thick.

8.4 Settlement

The roadway profile will be raised approximately 1.5 ft at the south abutment. Settlements due to the addition of this fill have been calculated to be less than ½ inch. Due to the granular nature of the subsurface soils present at the site all settlement associated with this fill occur will during construction having negligible effect on the finished bridge structure. Any settlement of the bridge abutments will be due to the elastic compression of the piling and will be negligible. See Appendix C- Calculations at the end of this report for supporting documentation.

8.5 Frost Protection

Any foundation placed on granular subgrade soils should be designed with an appropriate embedment for frost protection. According to the MaineDOT BDG Design Freezing Index map (MaineDOT BDG Figure 5-1) the site has a design-freezing index of approximately 1800 F-degree days. In a granular soil with a water content of approximately 10%, this correlates to a frost depth of approximately 7.5 feet. Therefore, any foundations placed on granular soils should be founded a minimum of 7.5 feet below finished exterior grade for frost protection. This minimum embedment depth applies only to foundations placed on subgrade soils. Pile supported integral abutments shall be embedded a minimum of 4.0 feet for frost protection per Figure 5-2 of the MaineDOT BDG. See Appendix C- Calculations at the end of this report for supporting documentation.

8.6 Seismic Design Considerations

In conformance with LRFD Article 4.7.4.2 seismic analysis is not required for single-span bridges regardless of seismic zone. According to Figure 2-2 of the MaineDOT BDG, the Canal Street Bridge is not on the National Highway System (NHS). The bridge is not classified as a major structure since the construction costs will not exceed $10 million. These criteria eliminate the MaineDOT BDG requirement to design the foundations for seismic earth loads. However, superstructure connections and minimum support length requirements shall be satisfied per LRFD Articles 3.10.9 and 4.7.4.4, respectively.

The following parameters were determined for the site from the USGS Seismic Parameters CD provided with the LRFD manual and LRFD Articles 3.10.3.1 and 3.10.6:

- Peak Ground Acceleration coefficient (PGA) = 0.083g
- Site Class C (very dense soil with average N-values greater than 50 bpf)
- Acceleration coefficient (A_s) = 0.099
- Design spectral acceleration coefficient at 0.2-second period (S_{DS}) = 0.206g
- Design spectral acceleration coefficient at 1.0-second period (S_{D1}) = 0.082g
- Seismic Zone 1 (based on S_{D1} less than or equal to 0.15g)

See Appendix C- Calculations at the end of this report for supporting documentation.

8.7 Construction Considerations

If using pile supported integral abutments at the site, pile lengths at Abutment No. 1 will be short and, therefore, scour protection will be critical. For scour protection, the integral abutments should be moved away from the channel. Since the proposed bridge design will rely on the riprap slopes to provide scour protection for the integral abutment piles, slope construction and riprap placement are of critical importance. Care should be taken in construction of the riprap slopes to assure that they are constructed in accordance with MaineDOT Special Provisions 610 and 703 and the Plans.

Frequent boulders and cobbles were encountered within the existing fill at the site. There is potential for these obstructions to impact the pile driving and/or installation operations. Obstructions may be cleared by conventional excavation methods, pre-augering, pre-drilling, or down-hole hammers. Clearing obstructions shall be specified as incidental to related pay items. Care should be taken to drive piles within allowable tolerances. Alternative methods to clear obstructions may be used as approved by the Resident.

Construction of the abutments will require soil excavation and partial or full removal of the existing structure. Construction activities may require cofferdams and/or earth support systems. The removal of the existing structure may require the replacement of excavated soils with compacted granular fill prior to pile driving.

In some locations the native soils may be saturated and significant water seepage may be encountered during construction. There may be localized sloughing and surface instability in some soil slopes. The Contractor should control groundwater, surface water infiltration and soil erosion during construction.

Using the excavated native soils as structural backfill should not be permitted. The native soils may only be used as common borrow in accordance with MaineDOT Standard Specifications 203 and 703.

The Contractor will have to excavate the existing subbase and subgrade fill soils in the bridge approaches. These materials should not be used to re-base the new bridge approaches.

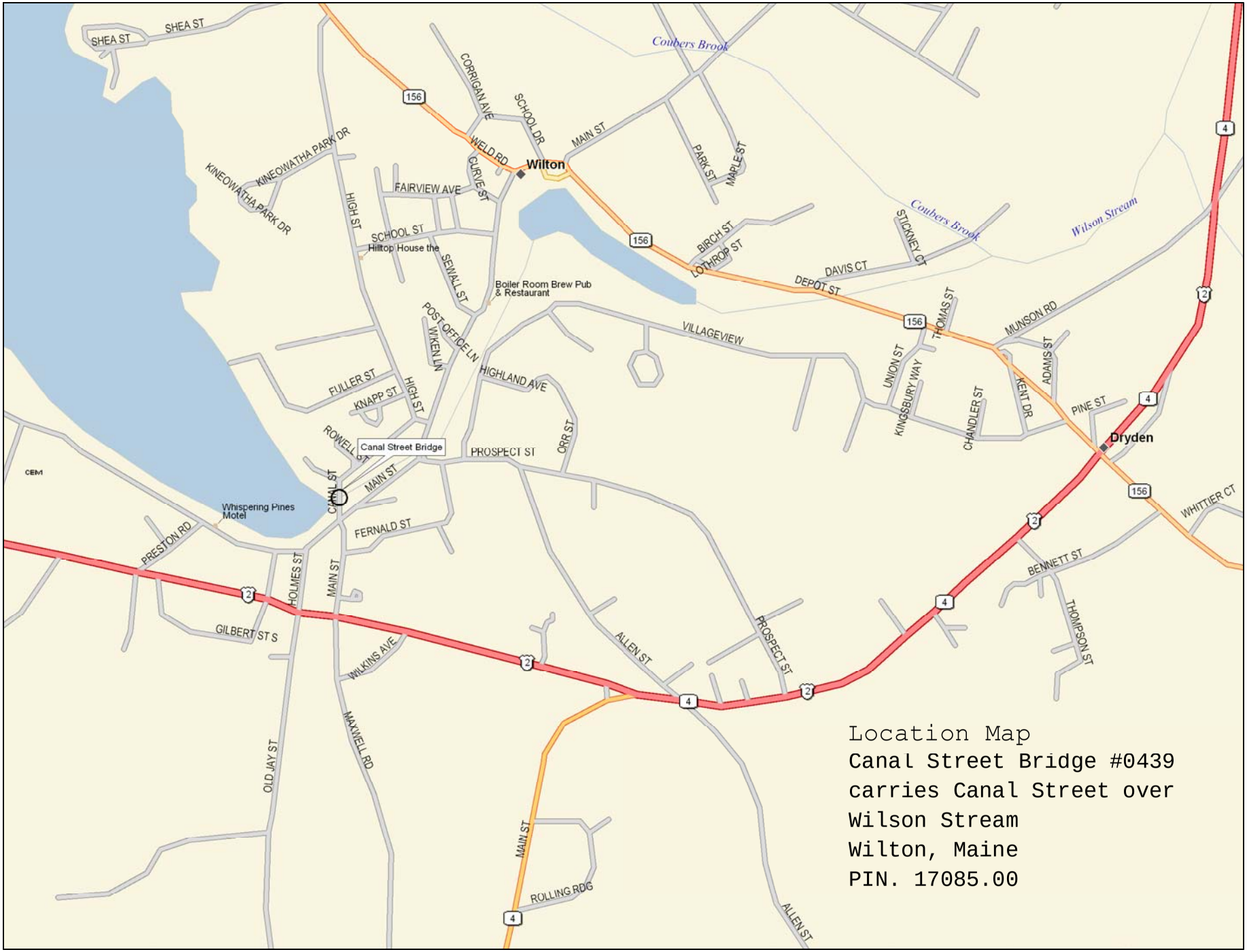
Excavated subbase sand and gravel may be used as fill below subgrade level in fill areas provided all other requirements of MaineDOT Standard Specifications 203 and 703 are met.

9.0 CLOSURE

This report has been prepared for the use of the MaineDOT Bridge Program for specific application to the proposed replacement of the Canal Street Bridge in Wilton in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is implied. In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. Further, the analyses and recommendations are based in part upon limited soil explorations at discrete locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

We also recommend that we be provided the opportunity for a general review of the final design plans and specifications in order to verify that the earthwork and foundation recommendations have been properly interpreted and implemented in the design.

Sheets



Location Map
Canal Street Bridge #0439
carries Canal Street over
Wilson Stream
Wilton, Maine
PIN. 17085.00

SHEET 1

Data use subject to license.
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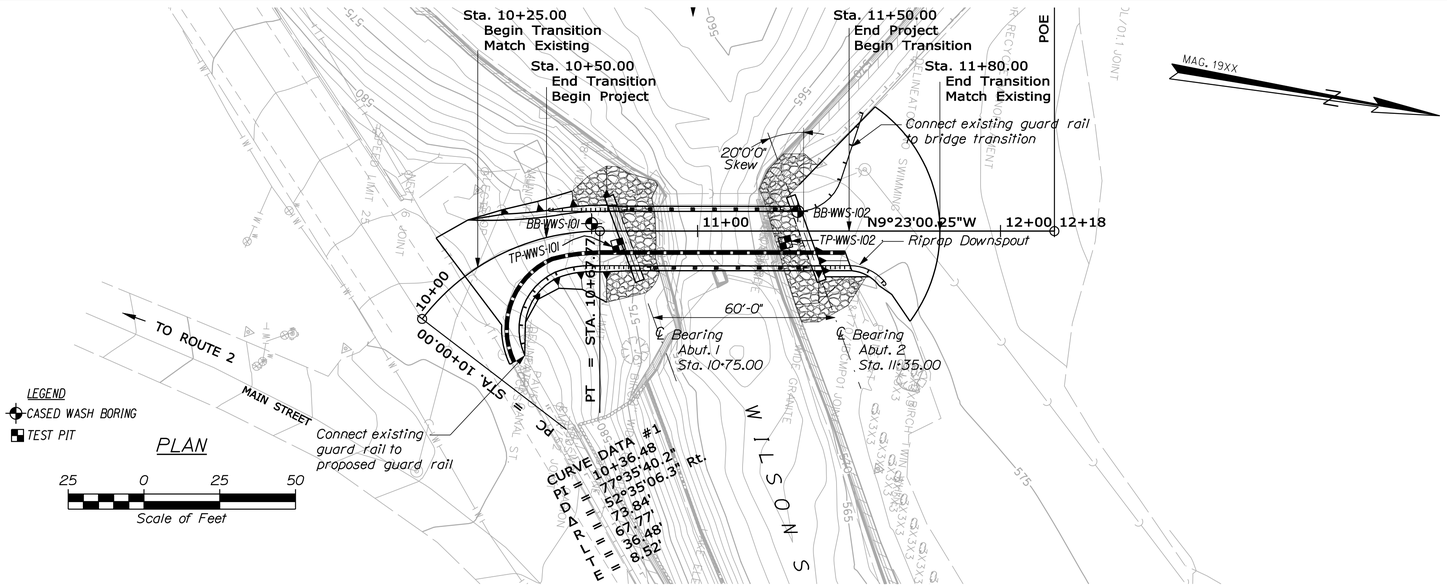
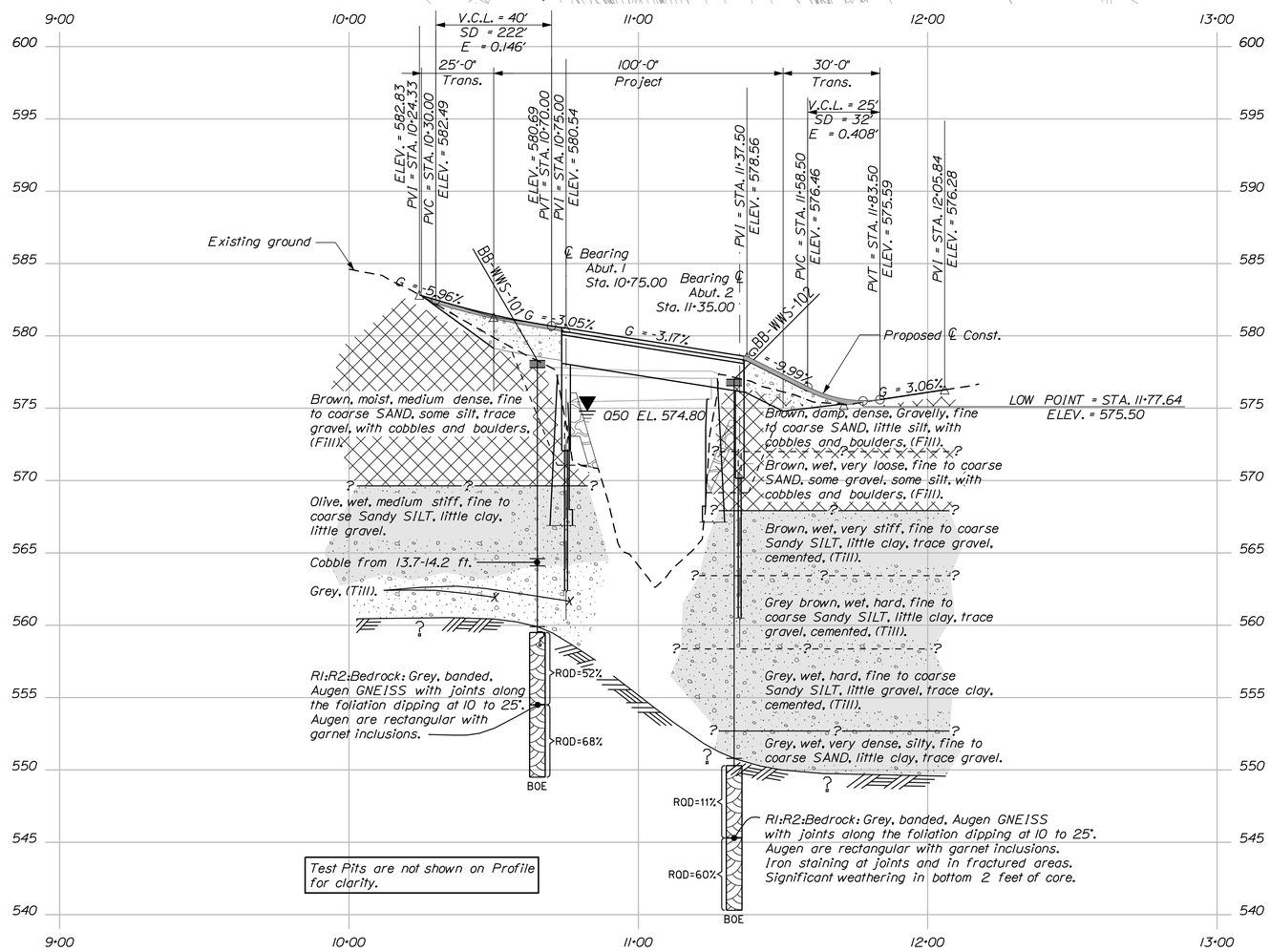
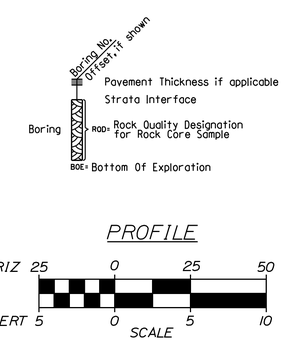
www.delorme.com

TN
MN (16.1°W)

Scale 1 : 12,800
0 200 400 600 800 1000 ft
0 100 200 300 400 500 m
1" = 1,066.7 ft
Data Zoom 14-0







Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1708(500)X

PIN 17085.00

BRIDGE NO. 0439

BRIDGE PLANS

			DATE
PROJ. MANAGER	N. BENOIT	BY	DATE
DESIGN-DETAILED	K.MAGUIRE	T.WHITE	OCT 2010
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	------	------	------
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

DATE

CANAL STREET BRIDGE
WILSON STREAM
WILTON FRANKLIN COUNTY

BORING LOCATION PLAN &
INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
2
OF 3

Filename: ...\GEOTECH\MSTA\006_BLP&ISP1.dgn Division: GEOTECH Username: terry.white Date:12/13/2010

Filename: ...\007_TEST PIT & BORING LOGS1.dgn Division: GEOTECH Username: terry.white Date:12/13/2010

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 over Wilson Stream
Location: Wilton, Maine

Boring No.: BB-WWS-101
PIN: 17085.00

Driller:	MaineDOT	Elevation (ft.)	578.3	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Doggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/5/10; 07:00-13:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	10+65.1, 2.5 Lt.	Casing ID/OD:	HW & NW	Water Level*:	9.0 ft bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐	Rope & Cathead ☐	

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test. PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WOP = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = Hammer Efficiency Factor/60%*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	577.80	Pavement — 0.50	
5	1D	24/16	5.00 - 7.00	7/5/3/12	8	11	SPLIT CASE		Brown, moist, medium dense, fine to coarse SAND, some silt, trace gravel. (Fill).	G#240079 A-2-4, SM WC=10.6%
								569.80	— 8.50	
10	2D	24/18	9.00 - 11.00	4/2/2/4	4	6			Olive, wet, medium stiff, fine to coarse Sandy SILT, little clay, little gravel.	G#240080 A-4, CL-ML WC=16.4%
15							15		Cobble from 13.7-14.2 ft bgs. 14.2-18.4 ft bgs, grey TILL from wash water.	
							18		Drop in NW Casing at 17.0 ft bgs.	
							50		950 blows for 0.4 ft.	
	R1	60/60	18.80 - 23.80	RQD = 52%			950 NQ-2	559.90	— 18.40 Top of Bedrock at Elev. 559.9 ft. Roller Coned ahead to 18.8 ft bgs. Bedrock: Grey, banded, Augen Gneiss with joints along the foliation dipping at 10 to 25 degrees. The Augen are typically rectangular and may be Amphibole with inclusions of garnet. R1:Core Times (min:sec) 18.8-19.8 ft (2:45) 19.8-20.8 ft (2:20) 20.8-21.8 ft (2:10) 21.8-22.8 ft (2:00) 22.8-23.8 ft (2:10) 100% Recovery	
	R2	60/60	23.80 - 28.80	RQD = 68%					R2:Core Times (min:sec) 23.8-24.8 ft (2:00) 24.8-25.8 ft (1:40) 25.8-26.8 ft (2:20) 26.8-27.8 ft (2:20) 27.8-28.8 ft (2:00) 100% Recovery	
								549.50	— 28.80 Bottom of Exploration at 28.80 feet below ground surface.	
50										

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-WWS-101

Maine Department of Transportation
Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 over Wilson Stream
Location: Wilton, Maine

Boring No.: BB-WWS-102
PIN: 17085.00

Driller:	MaineDOT	Elevation (ft.)	577.0	Auger ID/OD:	5" Solid Stem
Operator:	Giguere/Giles/Doggett	Datum:	NAVD88	Sampler:	Standard Split Spoon
Logged By:	B. Wilder	Rig Type:	CME 45C	Hammer Wt./Fall:	140#/30"
Date Start/Finish:	10/4/10; 08:30-15:00	Drilling Method:	Cased Wash Boring	Core Barrel:	NQ-2"
Boring Location:	11+33.1, 6.5 Lt.	Casing ID/OD:	HW & NW	Water Level*:	9.0 ft bgs.
Hammer Efficiency Factor: 0.84		Hammer Type:	Automatic ☒ Hydraulic ☐	Rope & Cathead ☐	

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test. PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WOP = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = Hammer Efficiency Factor/60%*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in. Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0							SSA	576.55	Pavement — 0.45	
	1D	24/15	1.00 - 3.00	13/12/10/15	22	31			Brown, damp, dense, Gravelly, fine to coarse SAND, little silt. (Fill).	G#240081 A-1-b, SM WC=3.9%
5	2D	24/13	5.00 - 7.00	2/2/1/6	3	4	SPLIT CASE		Brown, wet, very loose, fine to coarse SAND, some gravel, some silt, trace clay. (Fill).	G#240082 A-2-4, SC-SM WC=12.4%
								568.00	— 9.00	
10	3D	24/10	9.00 - 11.00	5/6/7/11	13	18			Brown, wet, very stiff, fine to coarse Sandy SILT, little clay, trace gravel, cemented. (Till).	
								564.00	— 13.00	
15	4D	24/18	14.50 - 16.50	14/22/25/30	47	66	OPEN HOLE		Grey brown, wet, hard, fine to coarse Sandy SILT, little clay, trace gravel, cemented. (Till).	G#240083 A-4, CL-ML WC=9.9%
								559.00	— 18.00	
20	5D	24/20	19.00 - 21.00	15/22/37/43	59	83			Grey, wet, hard, fine to coarse Sandy SILT, little gravel, trace clay, cemented. (Till).	G#240084 A-4, SC-SM WC=9.7%
								553.00	— 24.00	
25	6D	24/20	24.00 - 26.00	12/23/35/50	58	81	50		Grey, wet, very dense, Silty, fine to coarse SAND, little clay, trace gravel.	G#240085 A-4, CL-ML WC=9.2%
							52			
	R1	60/60	26.70 - 31.70	RQD = 11%			NQ-2	550.80	— 26.20 Top of Bedrock at Elev. 550.8 ft. Roller Coned ahead from 26.2-26.7 ft bgs. Bedrock: Grey, banded, Augen Gneiss with joints along the foliation dipping at 10 to 25 degrees. The Augen are typically rectangular and may be Amphibole with inclusions of Garnet. Iron staining at joints and throughout the fractured areas. R1:Core Times (min:sec) 26.7-27.7 ft (3:50) 27.7-28.7 ft (3:30) 28.7-29.7 ft (2:15) 29.7-30.7 ft (2:00) 30.7-31.7 ft (3:50) 100% Recovery Bedrock: Similar to above with significant weathering and fragmentation in the bottom 2 feet of core. R2:Core Times (min:sec) 31.7-32.7 ft (1:30) 32.7-33.7 ft (1:40) 33.7-34.7 ft (1:50) 34.7-35.7 ft (1:55) 35.7-36.7 ft (2:05) 100% Recovery	
	R2	60/60	31.70 - 36.70	RQD = 60%						
								540.30	— 36.70 Bottom of Exploration at 36.70 feet below ground surface.	
50										

Remarks:

Dropped in NW Casing at 24.0 ft bgs.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Boring No.: BB-WWS-102

Maine Department of Transportation
Test Pit Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 Over Wilson Stream
Location: Wilton, Maine

Test Pit No.: TP-WWS-101
PIN: 17085.00

Contractor:	MaineDOT Jay Crew	Equipment Type:	CASE 580 Series M	Elevation (ft.):	578.3
Operator:	Bob B.	Sampling Method:	N/A	Datum:	NAVD88
Logged By:	K. Maguire	Test Pit Dimensions (ft):	8.0 x 10		
Date Start/Finish:	9/8/2010	Total Depth (ft):	8.5 ft		
Location:	Behind Abutment No. 1	Water Level* (ft):	8.5 ft bgs		

Definitions:
S = Grab Sample
V = Insitu Vane Shear Test
U = Thin Wall Tube Sample

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis

Depth (ft.)	Sample No.	Sample Depth (ft.)	Shear Strength (psf)	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						10" to 12" Pavement — 1.0	
3				577.30		Brown, damp, fine to coarse SAND, with frequent cobbles and boulders.	
				574.30		— 4.0	
				574.20		Approximately 2" layer of pavement at 4 ft bgs. — 4.1	
						Brown, damp, fine to coarse SAND, with frequent cobbles and boulders.(Fill)	
6						Bottom of abutment observed at depth of 6 feet below roadway surface. Abutment has no footing and is sitting on sand and boulder fill.	
						Brown, damp, fine to coarse SAND, with frequent cobbles and boulders. (Fill).	
						Water and rock fill prevent digging test pit any deeper than 8.5 feet.	
9				569.80		— 8.5 Bottom of Test Pit at 8.5 feet below ground surface.	
12							
15							
18							

Remarks:

Abutment wall batter = 3" in 12".
No change in wall batter at back of abutment with depth.
At abutment on Main Street side.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Test Pit No. #P-WWS-101

Maine Department of Transportation
Test Pit Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 Over Wilson Stream
Location: Wilton, Maine

Test Pit No.: TP-WWS-102
PIN: 17085.00

Contractor:	MaineDOT Jay Crew	Equipment Type:	CASE 580 Series M	Elevation (ft.):	577.0
Operator:	Bob B.	Sampling Method:	N/A	Datum:	NAVD88
Logged By:	K. Maguire	Test Pit Dimensions (ft):	8.0 x 10		
Date Start/Finish:	9/7/2010	Total Depth (ft):	8.2		
Location:	Behind Abutment No. 2	Water Level* (ft):	8.2 bgs		

Definitions:
S = Grab Sample
V = Insitu Vane Shear Test
U = Thin Wall Tube Sample

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis

Depth (ft.)	Sample No.	Sample Depth (ft.)	Shear Strength (psf)	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class
0						9" Pavement — 0.8	
				576.2			
3						Brown, moist, fine to coarse SAND, with rock fill. Rock fill consists of large, rounded cobbles and boulders. fill matrix has more cobbles and boulders than soil.	
6							
						Water and rock fill prevent digging test pit any deeper than 8.5 feet.	
						Bottom/foundation of abutment not observed in test pit.	
9				568.50		— 8.5 Bottom of Test Pit at 8.5 feet below ground surface.	
12							
15							
18							

Remarks:

Abutment wall batter = 3" in 12".
No change in wall batter at back of abutment with depth.
At abutment on boat ramp side.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Page 1 of 1
Test Pit No. #P-WWS-102

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

BH-1708(500)X

PIN 17085.00

BRIDGE NO. 0439

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	N. BENOIT		
DESIGN-DETAILED	K.MAGUIRE	T. WHITE	OCT 2010
CHECKED-REVIEWED	------	------	------
DESIGN2-DETAILED2	------	------	------
DESIGN3-DETAILED3	------	------	------
REVISIONS 1	------	------	------
REVISIONS 2	------	------	------
REVISIONS 3	------	------	------
REVISIONS 4	------	------	------
FIELD CHANGES	------	------	------

SIGNATURE

P.E. NUMBER

DATE

CANAL STREET BRIDGE
WILSON STREAM
WILTON FRANKLIN COUNTY

TEST PIT & BORING LOGS

SHEET NUMBER
3
OF 3

Appendix A

Test Pit
and
Boring Logs

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations: (in this order)

Color (Munsell color chart)
Moisture (dry, damp, moist, wet, saturated)
Density/Consistency (from above right hand side)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc., if applicable)
Cementation (weak, moderate, or strong, if applicable, ASTM D 2488)
Geologic Origin (till, marine clay, alluvium, etc.)
Unified Soil Classification Designation
Groundwater level

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Consistency is rated according to standard penetration resistance.

Modified Burmister System

Descriptive Term	Portion of Total
trace	0% - 10%
little	11% - 20%
some	21% - 35%
adjective (e.g. sandy, clayey)	36% - 50%

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value blows per foot	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily Penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnai
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$RQD = \frac{\text{sum of the lengths of intact pieces of core* > 100 mm}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD
Very Poor	<25%
Poor	26% - 50%
Fair	51% - 75%
Good	76% - 90%
Excellent	91% - 100%

Desired Rock Observations: (in this order)

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Lithology (igneous, sedimentary, metamorphic, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5, low angle - 5-35, mod. dipping - 35-55, steep - 55-85, vertical - 85-90)
- -spacing (very close - <5 cm, close - 5-30 cm, mod. close 30-100 cm, wide - 1-3 m, very wide >3 m)
- -tightness (tight, open or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: AASHTO Standard Specification for Highway Bridges 17th Ed. Table 4.4.8.1.2A
Recovery

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Sample Container Labeling Requirements:

PIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Maine Department of Transportation

Test Pit Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439
Over Wilson Stream
Location: Wilton, Maine

Test Pit No.: TP-WWS-101
PIN: 17085.00

Contractor:	MaineDOT Jay Crew	**Equipment Type:**	CASE 580 Series M	**Elevation (ft.)**	578.3
Operator:	Bob B.	**Sampling Method:**	N/A	**Datum:**	NAVD88
Logged By:	K. Maguire	**Test Pit Dimensions (ft):**	10 x 10		
Date Start/Finish:	9/8/2010	**Total Depth (ft):**	8.5 ft		
Location:	Behind Abutment No. 1	**Water Level* (ft):**	8.5 ft bgs		

Definitions:
S = Grab Sample
V = Insitu Vane Shear Test
U = Thin Wall Tube Sample

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis

Depth (ft.)	Sample Information: Sample No.	Sample Depth (ft.)	Shear Strength (psf)	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						10" to 12" Pavement	
				577.30		1.0	
						Brown, damp, fine to coarse SAND, with frequent cobbles and boulders.	
3							
				574.30		4.0	
				574.20		Approximately 2" layer of pavement at 4 ft bgs.	
						4.1	
						Brown, damp, fine to coarse SAND, with fequent cobbles and boulders,(Fill)	
6						Bottom of abutment observed at depth of 6 feet below roadway surface. Abutment has no footing and is sitting on sand and boulder fill.	
						Brown, damp, fine to coarse SAND, with frequent cobbles and boulders, (Fill).	
						Water and rock fill prevent digging test pit any deeper than 8.5 feet.	
				569.80		8.5	
9						**Bottom of Test Pit at 8.5 feet below ground surface.**	
12							
15							
18							

Remarks:

Abutment wall batter = 3" in 12".
No change in wall batter at back of abutment with depth.
At abutment on Main Street side.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Test Pit Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439
Over Wilson Stream
Location: Wilton, Maine

Test Pit No.: TP-WWS-102
PIN: 17085.00

Contractor:	MaineDOT Jay Crew	**Equipment Type:**	CASE 580 Series M	**Elevation (ft.)**	577.0
Operator:	Bob B.	**Sampling Method:**	N/A	**Datum:**	NAVD88
Logged By:	K. Maguire	**Test Pit Dimensions (ft):**	10 x 10		
Date Start/Finish:	9/7/2010	**Total Depth (ft):**	8.2		
Location:	Behind Abutment No. 2	**Water Level* (ft):**	8.2 bgs		

Definitions:
S = Grab Sample
V = Insitu Vane Shear Test
U = Thin Wall Tube Sample

Definitions:
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)

Definitions:
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis

Depth (ft.)	Sample No.	Sample Depth (ft.)	Shear Strength (psf)	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0						9" Pavement	
				576.25		0.8	
3						Brown, moist, fine to coarse SAND, with rock fill. Rock fill consists of large, rounded cobbles and boulders. Fill matix has more cobbles and boulders than soil.	
6						Water and rock fill prevent digging test pit any deeper than 8.5 feet. Bottom/foundation of abutment not observed in test pit.	
				568.50		8.5	
9						**Bottom of Test Pit at 8.5 feet below ground surface.**	
12							
15							
18							

Remarks:

Abutment wall batter = 3" in 12".
No change in wall batter at back of abutment with depth.
At abutment on boat ramp side.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 over Wilson Stream
Location: Wilton, Maine

Boring No.: BB-WWS-101
PIN: 17085.00

Driller:	MaineDOT	**Elevation (ft.)**	578.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/5/10; 07:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+65.1, 2.5 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	577.80	Pavement — 0.50	
5	1D	24/16	5.00 - 7.00	7/5/3/12	8	11	SPUN CASE		Brown, moist, medium dense, fine to coarse SAND, some silt, trace gravel, (Fill).	G#240079 A-2-4, SM WC=10.6%
								569.80	8.50	
	2D	24/18	9.00 - 11.00	4/2/2/4	4	6			Olive, wet, medium stiff, fine to coarse Sandy SILT, little clay, little gravel.	G#240080 A-4, CL-ML WC=16.4%
10										
									Cobble from 13.7-14.2 ft bgs. 14.2-18.4 ft bgs, grey TILL from wash water.	
15							15			
							18			
							50		Drop in NW Casing at 17.0 ft bgs.	
	R1	60/60	18.80 - 23.80	RQD = 52%			[a]50 NQ-2	559.90	[a]50 blows for 0.4 ft. — 18.40	
20									Top of Bedrock at Elev. 559.9 ft. Roller Coned ahead to 18.8 ft bgs. Bedrock: Grey, banded, Augen Gneiss with joints along the foliation dipping at 10 to 25 degrees. The Augen are typically rectangular and may be Amphibole with inclusions of garnet. R1:Core Times (min:sec) 18.8-19.8 ft (2:45) 19.8-20.8 ft (2:20) 20.8-21.8 ft (2:10) 21.8-22.8 ft (2:00) 22.8-23.8 ft (2:10) 100% Recovery	
	R2	60/60	23.80 - 28.80	RQD = 68%					R2:Core Times (min:sec) 23.8-24.8 ft (2:00)	
25										

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-WWS-101

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 over Wilson Stream
Location: Wilton, Maine

Boring No.: BB-WWS-101
PIN: 17085.00

Driller:	MaineDOT	**Elevation (ft.)**	578.3	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/5/10; 07:00-13:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	10+65.1, 2.5 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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V = Insitu Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Insitu Vane Shear Test attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										24.8-25.8 ft (1:40) 25.8-26.8 ft (2:20) 26.8-27.8 ft (2:20) 27.8-28.8 ft (2:00) 100% Recovery	
								549.50		28.80 **Bottom of Exploration at 28.80 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 over Wilson Stream
Location: Wilton, Maine

Boring No.: BB-WWS-102
PIN: 17085.00

Driller:	MaineDOT	**Elevation (ft.)**	577.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/4/10; 08:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	11+33.1, 6.5 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = weight of 140lb. hammer
WOR/C = weight of rods or casing
WO1P = Weight of one person
S_u = Insitu Field Vane Shear Strength (psf)
T_v = Pocket Torvane Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Sample Information

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	576.55	Pavement — 0.45	
	1D	24/15	1.00 - 3.00	13/12/10/15	22	31			Brown, damp, dense, Gravelly, fine to coarse SAND, little silt, (Fill).	G#240081 A-1-a, SM WC=3.9%
5	2D	24/13	5.00 - 7.00	2/2/1/6	3	4	SPUN CASE		Brown, wet, very loose, fine to coarse SAND, some gravel, some silt, trace clay, (Fill).	G#240082 A-2-4, SC-SM WC=12.4%
								568.00	9.00	
10	3D	24/10	9.00 - 11.00	5/6/7/11	13	18			Brown, wet, very stiff, fine to coarse Sandy SILT, little clay, trace gravel cemented, (Till).	
								564.00	13.00	
15	4D	24/18	14.50 - 16.50	14/22/25/30	47	66	OPEN HOLE		Grey brown, wet, hard, fine to coarse Sandy SILT, little clay, trace gravel, cemented, (Till).	G#240083 A-4, CL-ML WC=9.9%
								559.00	18.00	
20	5D	24/20	19.00 - 21.00	15/22/37/43	59	83			Grey, wet, hard, fine to coarse Sandy SILT, little gravel, trace clay, cemented, (Till).	G#240084 A-4, SC-SM WC=9.7%
								553.00	24.00	
25	6D	24/20	24.00 - 26.00	12/23/35/50	58	81	50		Grey, wet, very dense, Silty, fine to coarse SAND, little clay, trace	G#240085 A-4, CL-ML

Remarks:

Dropped in NW Casing at 24.0 ft bgs.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Boring No.: BB-WWS-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Canal Street Bridge #0439 over Wilson Stream
Location: Wilton, Maine

Boring No.: BB-WWS-102
PIN: 17085.00

Driller:	MaineDOT	**Elevation (ft.)**	577.0	**Auger ID/OD:**	5" Solid Stem
Operator:	Giguere/Giles/Daggett	**Datum:**	NAVD88	**Sampler:**	Standard Split Spoon
Logged By:	B. Wilder	**Rig Type:**	CME 45C	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	10/4/10; 08:30-15:00	**Drilling Method:**	Cased Wash Boring	**Core Barrel:**	NQ-2"
Boring Location:	11+33.1, 6.5 Lt.	**Casing ID/OD:**	HW & NW	**Water Level*:**	9.0 ft bgs.
Hammer Efficiency Factor:	0.84	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample attempt
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SSA = Solid Stem Auger
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T_v = Pocket Torvane Shear Strength (psf)
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N-uncorrected = Raw field SPT N-value
Hammer Efficiency Factor = Annual Calibration Value
N_{60} = SPT N-uncorrected corrected for hammer efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
$S_{u(lab)}$ = Lab Vane Shear Strength (psf)
WC = water content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25							52			gravel.	WC=9.2%
	R1	60/60	26.70 - 31.70	RQD = 11%			NQ-2	550.80		26.20 Top of Bedrock at Elev. 550.8 ft. Roller Coned ahead from 26.2-26.7 ft bgs. Bedrock: Grey, banded, Augen Gneiss with joints along the foliation dipping at 10 to 25 degrees. The Augen are typically rectangular and may be Amphibole with inclucions of Garnet. Iron staining at joints and throughout the fractured areas. R1:Core Times (min:sec) 26.7-27.7 ft (3:50) 27.7-28.7 ft (3:30) 28.7-29.7 ft (2:15) 29.7-30.7 ft (2:00) 30.7-31.7 ft (3:50) 100% Recovery	
30											
	R2	60/60	31.70 - 36.70	RQD = 60%						Bedrock: Similar to above with significant weathering and fragmentation in the bottom 2 feet of core. R2:Core Times (min:sec) 31.7-32.7 ft (1:30) 327.7-33.7 ft (1:40) 33.7-34.7 ft (1:50) 34.7-35.7 ft (1:55) 35.7-36.7 ft (2:05) 100% Recovery	
35											
								540.30		36.70 **Bottom of Exploration at 36.70 feet below ground surface.**	
40											
45											
50											

Remarks:

Dropped in NW Casing at 24.0 ft bgs.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Appendix B

Laboratory Data

State of Maine - Department of Transportation

Laboratory Testing Summary Sheet

Town(s): Wilton Project Number: 17085.00

Boring & Sample Identification Number	Station (Feet)	Offset (Feet)	Depth (Feet)	Reference Number	G.S.D.C. Sheet	W.C. %	L.L.	P.I.	Classification		
									Unified	AASHTO	Frost
BB-WWS-101, 1D	10+65.1	2.5 Lt.	5.0-7.0	240079	1	10.6			SM	A-2-4	II
BB-WWS-101, 2D	10+65.1	2.5 Lt.	9.0-11.0	240080	1	16.4			CL-ML	A-4	IV
BB-WWS-102, 1D	11+33.1	6.5 Lt.	1.0-3.0	240081	2	3.9			SM	A-1-a	II
BB-WWS-102, 2D	11+33.1	6.5 Lt.	5.0-7.0	240082	2	12.4			SC-SM	A-2-4	II
BB-WWS-102, 4D	11+33.1	6.5 Lt.	14.5-16.5	240083	2	9.9			CL-ML	A-4	IV
BB-WWS-102, 5D	11+33.1	6.5 Lt.	19.0-21.0	240084	2	9.7			SC-SM	A-4	III
BB-WWS-102, 6D	11+33.1	6.5 Lt.	24.0-26.0	240085	2	9.2			CL-ML	A-4	IV

Classification of these soil samples is in accordance with AASHTO Classification System M-145-40. This classification is followed by the "Frost Susceptibility Rating" from zero (non-frost susceptible) to Class IV (highly frost susceptible). The "Frost Susceptibility Rating" is based upon the MaineDOT and Corps of Engineers Classification Systems.

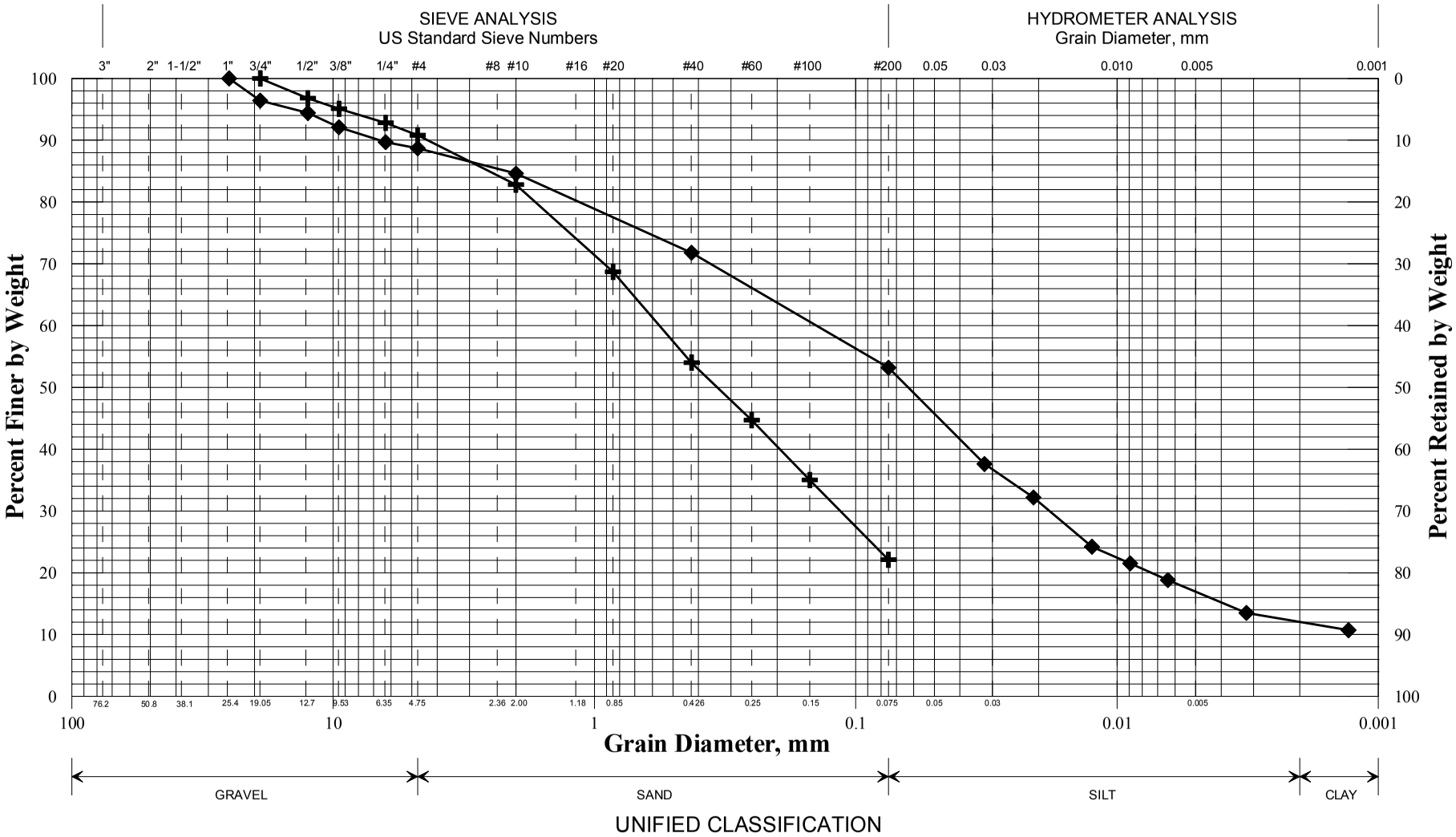
GSDC = Grain Size Distribution Curve as determined by AASHTO T 88-93 (1996) and/or ASTM D 422-63 (Reapproved 1998)

WC = water content as determined by AASHTO T 265-93 and/or ASTM D 2216-98

LL = Liquid limit as determined by AASHTO T 89-96 and/or ASTM D 4318-98

PI = Plasticity Index as determined by AASHTO 90-96 and/or ASTM D4318-98

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



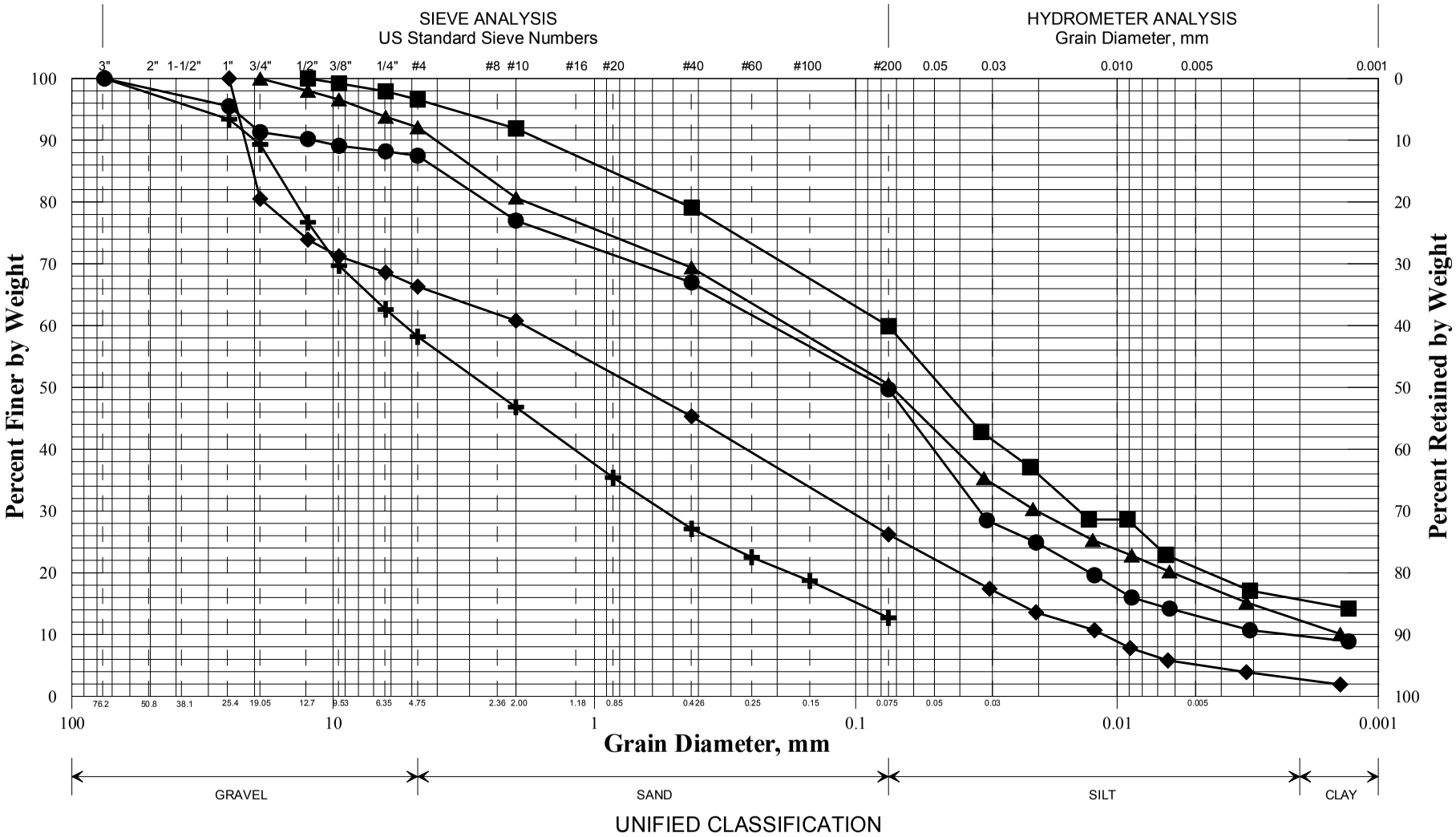
UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-WWS-101/1D	10+65.1	2.5 LT	5.0-7.0	SAND, some silt, trace gravel.	10.6			
◆	BB-WWS-101/2D	10+65.1	2.5 LT	9.0-11.0	Sandy SILT, little clay, little gravel.	16.4			
■									
●									
▲									
✕									

PIN
017085.00
Town
Wilton
Reported by/Date
WHITE, TERRY A 11/9/2010

SHEET 1

State of Maine Department of Transportation
GRAIN SIZE DISTRIBUTION CURVE



UNIFIED CLASSIFICATION

	Boring/Sample No.	Station	Offset, ft	Depth, ft	Description	W, %	LL	PL	PI
✚	BB-WWS-102/1D	11+33.1	6.5 LT	1.0-3.0	Gravelly SAND, little silt.	3.9			
◆	BB-WWS-102/2D	11+33.1	6.5 LT	5.0-7.0	SAND, some gravel, some silt, trace clay.	12.4			
■	BB-WWS-102/4D	11+33.1	6.5 LT	14.5-16.5	Sandy SILT, little clay, trace gravel.	9.9			
●	BB-WWS-102/5D	11+33.1	6.5 LT	19.0-21.0	Sandy SILT, little gravel, trace clay.	9.7			
▲	BB-WWS-102/6D	11+33.1	6.5 LT	24.0-26.0	Silty SAND, little clay, trace gravel.	9.2			
✕									

PIN
017085.00
Town
Wilton
Reported by/Date
WHITE, TERRY A 11/12/2010

Appendix C

Calculations

Abutment Foundations: Integral Driven H-piles

Axial Structural Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

Look at the following piles:

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

H-pile Steel area: $A_s := \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

yield strength: $F_y := 50 \cdot ksi$

Determine equivalent yield resistance $P_o = QF_yA_s$ LRFD Article 6.9.4.1.1

$Q := 1.0$ LRFDArticle 6.9.4.2 $F_y = 50 \cdot ksi$

$P_o := Q \cdot F_y \cdot A_s$

$$P_o = \begin{pmatrix} 775 \\ 1090 \\ 1070 \\ 1305 \\ 1720 \end{pmatrix} \cdot kip$$

Determine elastic critical buckling resistance: $Pe = \pi^2 EA_s/(Kl/r_s)^2$ LRFD eq. 6.9.4.1.2-1

E = steel modulus $E := 29000 \cdot ksi$

K = effective length factor $K_{eff} := 2.0$ LRFD Table C4.6.2.5-1 (assume fixed head, and rotation free, translation fixed at tip)

l = unbraced length $l_{unbraced} := 48 \cdot in$

r_s = radius of gyration $r_s := \begin{pmatrix} 2.86 \\ 2.92 \\ 3.49 \\ 3.53 \\ 3.59 \end{pmatrix} \cdot in$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

LRFD Article C6.9.4.1.2 states that the critical flexural buckling resistances be calculayed about the x- and y-axes with the smaller value taken as Pe.

Use y-axis as this results in the smaller value.

LRFD eq. 6.9.4.1.2-1

$$P_e := \overrightarrow{\left[\frac{\pi^2 \cdot E}{\left(\frac{K_{eff} \cdot l_{unbraced}}{r_s} \right)^2} \cdot A_s \right]}$$

$$P_e = \begin{pmatrix} 3937 \\ 5773 \\ 8095 \\ 10101 \\ 13769 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

LRFD Article 6.9.4.1.1

$$\frac{P_e}{P_o} = \begin{pmatrix} 5.0806 \\ 5.296 \\ 7.5655 \\ 7.7399 \\ 8.0052 \end{pmatrix}$$

If Pe/Po> or = 0.44 then:

LRFD Equation 6.9.4.1.1-1

$$P_n := \left[\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \cdot P_o\right]$$

$$P_n = \begin{pmatrix} 714 \\ 1007 \\ 1012 \\ 1236 \\ 1632 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Resistance:

Driving conditions are assumed "severe" due to presence of cobbles and boulders.

Strength Limit State Axial Resistance factor for piles in compression under severe driving conditions:

From Article 6.5.4.2 $\phi_c := 0.5$

Factored Compressive Resistance: eq. 6.9.2.1-1

$$P_r := \phi_c \cdot P_n \qquad P_r = \begin{pmatrix} 357 \\ 504 \\ 506 \\ 618 \\ 816 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Service and Extreme Limit States Axial Resistance

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.8.3

$\phi := 1.0$

Factored Compressive Resistance for Service and Extreme Limit States:

eq. 6.9.2.1-1

$$P_r := \phi \cdot P_n \qquad P_r = \begin{pmatrix} 714 \\ 1007 \\ 1012 \\ 1236 \\ 1632 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

Geotechnical Resistance

Assume piles will be end bearing on bedrock driven through overlying fill, sandy silt and silty sand.

Bedrock Type:

Gneiss RQD ranges from 11 to 68%

Use RQD = 50% and ϕ = 27 to 34 deg (LRFD Table C10.4.6.4-1)

Axial Geotechnical Resistance of H-piles

Ref: AASHTO LRFD Bridge Design Specifications 5th Edition 2010

Look at these piles:

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Note: All matrices set up in this order

Steel area: $A_s = \begin{pmatrix} 15.5 \\ 21.8 \\ 21.4 \\ 26.1 \\ 34.4 \end{pmatrix} \cdot in^2$

Pile depth: $d := \begin{pmatrix} 11.78 \\ 12.13 \\ 13.61 \\ 13.83 \\ 14.21 \end{pmatrix} \cdot in$

Pile width: $b := \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

End bearing resistance of piles on bedrock - LRFD code specifies Canadian Geotech Method 1985 (LRFD Table 10.5.5.2.3-1) Canadian Foundation Manual 4th Edition (2006) Section 18.6.3.3.

Average compressive strength of rock core
from AASHTO Standard Spec for Highway Bridges 17 Ed.
Table 4.4.8.1.2B pg 64

q_u for sandstone compressive strength ranges from 9700 to 25000 psi

use $\sigma_c := 18000 \cdot psi$

Determine K_{sp}: From Canadian Foundation Manual 4th Edition (2006) Section 9.2

Spacing of discontinuities: $c := 48 \cdot in$ Assumed based on rock core

Aperture of discontinuities: $\delta := \frac{1}{64} \cdot in$ joints are tight

Footing width, b: $b = \begin{pmatrix} 12.045 \\ 12.215 \\ 14.585 \\ 14.695 \\ 14.885 \end{pmatrix} \cdot in$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

$$K_{sp} := \frac{3 + \frac{c}{b}}{10 \cdot \left(1 + 300 \cdot \frac{\delta}{c}\right)^{0.5}}$$

$K_{sp} = \begin{pmatrix} 0.6667 \\ 0.6614 \\ 0.6005 \\ 0.5981 \\ 0.5941 \end{pmatrix}$

K_{sp} includes a factor of safety of 3

Length of rock socket, L_s: $L_s := 0 \cdot in$ Pile is end bearing on rock

Diameter of socket, B_s: $B_s := 1 \cdot ft$

depth factor, d_f: $d_f := 1 + 0.4\left(\frac{L_s}{B_s}\right)$ $d_f = 1$ should be < or = 3 OK

$$q_a := \sigma_c \cdot K_{sp} \cdot d_f \qquad q_a = \begin{pmatrix} 1728 \\ 1714 \\ 1556 \\ 1550 \\ 1540 \end{pmatrix} \cdot ksf$$

Nominal Geotechnical Tip Resistance, R_p:

Multiply by 3 to take out FS=3 on K_{sp}

$$R_p := \overrightarrow{\left(3q_a \cdot A_s\right)} \qquad R_p = \begin{pmatrix} 558 \\ 779 \\ 694 \\ 843 \\ 1104 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

STRENGTH LIMIT STATE:

Factored Geotechnical Resistance at Strength Limit State:

Resistance factor, end bearing on rock (Canadian Geotech. Society, 1985 method):

Nominal resistance of Single Pile in Axial Compression - Static Analysis Methods, ϕ_{stat} $\phi_{stat} := 0.45$ LRFD Table 10.5.5.2.3-1

$$R_f := \phi_{stat} \cdot R_p \qquad R_f = \begin{pmatrix} 251 \\ 350 \\ 312 \\ 379 \\ 497 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Strength Limit State

SERVICE/EXTREME LIMIT STATES:

Factored Geotechnical Resistance at the Service/Extreme Limit States:

Resistance Factors for Service and Extreme Limit States ϕ = 1.0 LRFD 10.5.5.1 and 10.5.5.3

$$\phi := 1.0$$

$$R_{fse} := \phi \cdot R_p \qquad R_{fse} = \begin{pmatrix} 558 \\ 779 \\ 694 \\ 843 \\ 1104 \end{pmatrix} \cdot kip$$

HP 12 x 53
HP 12 x 74
HP 14 x 73
HP 14 x 89
HP 14 x 117

Service/Extreme Limit States

DRIVABILITY ANALYSIS

Ref: LRFD Article 10.7.8

For steel piles in compression or tension
$\sigma_{dr} = 0.9 \times \phi_{da} \times f_y$ (eq. 10.7.8-1)

$f_y := 50 \cdot ksi$ yield strength of steel

$\phi_{da} := 1.0$ resistance factor from LRFD Table 10.5.5.2.3-1
Pile Drivability Analysis, Steel piles

$\sigma_{dr} := 0.9 \cdot \phi_{da} \cdot f_y$ $\sigma_{dr} = 45 \cdot ksi$ driving stresses in pile can not exceed 45 ksi

Compute Resistance that can be achieved in a drivability analysis:

The resistance that must be achieved in a drivability analysis will be the maximum applied pile axial load (must be less than the the factored geotechnical resistance from above as this governs)
divided by the appropriate resistance factor for wave equation analysis and dynamic test which will be required for construction.

Table 10.5.5.2.3-1 pg 10-45 gives resistance factor for dynamic test, ϕ_{dyn}:

$\phi_{dyn} := 0.65$

Pile Size = 12 x 53

Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation — 02-Dec-2010
Wilton Canal St Bridge Drivability 12x53 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
450.0	44.73	2.07	9.4	6.70	11.10
451.0	44.89	2.08	9.4	6.70	11.14
452.0	44.90	2.09	9.4	6.71	11.14
453.0	44.94	2.10	9.5	6.72	11.14
454.0	45.08	2.11	9.5	6.72	11.18
455.0	45.09	2.12	9.5	6.73	11.19
456.0	45.13	2.14	9.6	6.74	11.19
457.0	45.16	2.15	9.6	6.74	11.20
458.0	45.31	2.16	9.6	6.75	11.23
459.0	45.35	2.17	9.7	6.76	11.24

Limit driving stress to 45 ksi

Strength Limit State: $\phi_{dyn} = 0.65$

$R_{dr_12x53_strength} := 454 \cdot kip \cdot \phi_{dyn}$

$R_{dr_12x53_strength} = 295 \cdot kip$

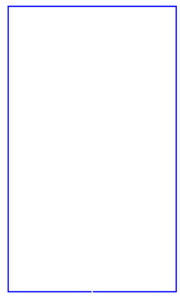
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_12x53_servext} := 454 \cdot kip$

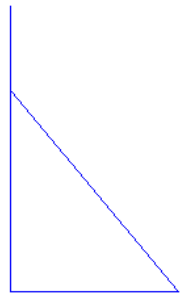
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	17.00	ft
Pile Penetration	12.00	ft
Pile Top Area	15.50	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 12 x 74

Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation — 02-Dec-2010
Wilton Canal St Bridge Drivability 12x74 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
555.0	44.95	2.59	6.8	9.26	17.44
556.0	44.84	2.56	6.8	9.27	17.40
557.0	44.88	2.57	6.8	9.27	17.42
558.0	44.97	2.58	6.8	9.28	17.44
559.0	45.08	2.59	6.9	9.28	17.46
560.0	45.13	2.58	6.9	9.29	17.47
561.0	45.22	2.60	6.9	9.30	17.49
562.0	45.30	2.61	6.9	9.30	17.51
563.0	45.21	2.59	6.9	9.31	17.46
564.0	45.33	2.60	7.0	9.31	17.49

Limit driving stress to 45 ksi

Strength Limit State: $\phi_{dyn} = 0.65$

$R_{dr_12x74_strength} := 559 \cdot kip \cdot \phi_{dyn}$

$R_{dr_12x74_strength} = 363 \cdot kip$

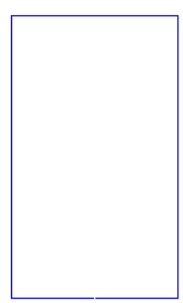
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_12x74_servext} := 559 \cdot kip$

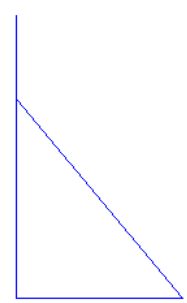
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	17.00	ft
Pile Penetration	12.00	ft
Pile Top Area	21.80	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 73

Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation — 02-Dec-2010
Wilton Canal St Bridge Drivability 14x73 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
540.0	44.64	2.66	6.6	9.19	17.31
541.0	44.73	2.67	6.6	9.20	17.33
542.0	44.81	2.69	6.6	9.21	17.35
543.0	44.87	2.69	6.6	9.21	17.38
544.0	45.00	2.71	6.6	9.22	17.40
545.0	44.92	2.69	6.7	9.22	17.35
546.0	44.98	2.70	6.7	9.23	17.37
547.0	45.13	2.71	6.7	9.25	17.44
548.0	45.24	2.73	6.7	9.25	17.46
549.0	45.12	2.71	6.7	9.26	17.41

Limit driving stress to 45 ksi

Strength Limit State: $\phi_{dyn} = 0.65$

$R_{dr_14x73_strength} := 544 \cdot kip \cdot \phi_{dyn}$

$R_{dr_14x73_strength} = 354 \cdot kip$

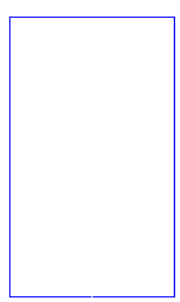
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_14x73_servext} := 544 \cdot kip$

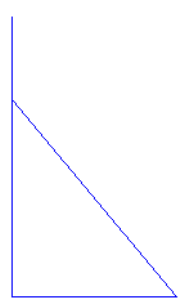
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	17.00	ft
Pile Penetration	12.00	ft
Pile Top Area	21.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 89

Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation — 02-Dec-2010
Wilton Canal St Bridge Drivability 14x89 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
700.0	44.91	2.76	9.1	9.85	17.91
701.0	44.96	2.77	9.1	9.85	17.91
702.0	44.98	2.77	9.1	9.86	17.92
703.0	44.99	2.77	9.2	9.87	17.92
704.0	45.08	2.80	9.2	9.86	17.92
705.0	45.07	2.79	9.2	9.87	17.93
706.0	45.11	2.80	9.2	9.88	17.93
707.0	45.28	2.83	9.2	9.88	17.99
708.0	45.31	2.84	9.3	9.89	17.99
709.0	45.36	2.84	9.3	9.89	18.00

Limit driving stress to 45 ksi

Strength Limit State: $\phi_{dyn} = 0.65$

$R_{dr_14x89_strength} := 703 \cdot kip \cdot \phi_{dyn}$

$R_{dr_14x89_strength} = 457 \cdot kip$

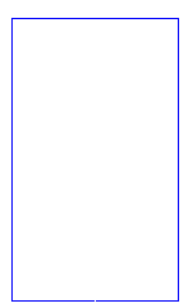
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_14x89_servext} := 703 \cdot kip$

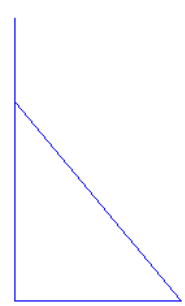
DELMAG D 19-42

Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	17.00	ft
Pile Penetration	12.00	ft
Pile Top Area	26.10	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Pile Size = 14 x 117

Assume Contractor will use a Delmag D19-42 hammer to install piles

State of Maine Dept. Of Transportation — 02-Dec-2010
Wilton Canal S Bridge Drivability 14x117 — GRLWEAP (TM) Version 2003

Ultimate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count blows/in	Stroke feet	Energy kips-ft
970.0	44.58	5.37	14.8	10.81	18.57
971.0	44.61	5.37	14.8	10.81	18.56
972.0	44.63	5.41	14.8	10.81	18.61
973.0	44.63	5.41	14.9	10.81	18.60
974.0	44.64	5.41	14.9	10.81	18.59
975.0	44.66	5.41	14.9	10.81	18.58
976.0	44.67	5.41	15.0	10.81	18.57
977.0	44.69	5.41	15.1	10.81	18.55
978.0	44.70	5.41	15.1	10.81	18.55
979.0	44.71	5.41	15.2	10.81	18.53

Limit to blow count to 15 blows per inch

Strength Limit State: $\phi_{dyn} = 0.65$

$R_{dr_14x117_strength} := 976 \cdot kip \cdot \phi_{dyn}$

$R_{dr_14x117_strength} = 634 \cdot kip$

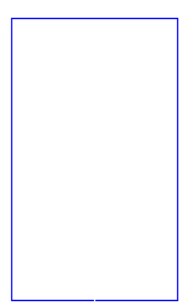
Service and Extreme Limit States: $\phi := 1.0$

$R_{dr_14x117_servext} := 976 \cdot kip$

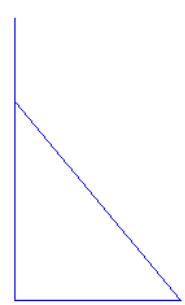
DELMAG D 19-42

Stroke	10.81	feet
Efficiency	0.800	
Helmet	3.20	kips
Hammer Cushion	109975	kips/in
Skin Quake	0.100	in
Toe Quake	0.040	in
Skin Damping	0.050	sec/ft
Toe Damping	0.150	sec/ft
Pile Length	17.00	ft
Pile Penetration	12.00	ft
Pile Top Area	34.40	in2

Pile Model



Skin Friction Distribution



Res. Shaft = 10 %
(Proportional)

Earth Pressure:

Passive Earth Pressure - Coulomb Theory
from Maine DOT Bridge Design Guide Section 3.6.6 pg 3-8

Angle of back face of wall to the horizontal: $\alpha := 90 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

Friction angle between fill and wall:
From LRFD Table 3.11.5.3-1 range from 17 to 22 $\delta := 20 \cdot deg$

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

$$K_p := \frac{\sin(\alpha - \phi)^2}{\sin(\alpha)^2 \cdot \sin(\alpha + \delta) \cdot \left(1 - \sqrt{\frac{\sin(\phi + \delta) \cdot \sin(\phi + \beta)}{\sin(\alpha + \delta) \cdot \sin(\alpha + \beta)}}\right)^2}$$

$K_p = 6.89$

Passive Earth Pressure - Rankine Theory
from Bowles 5th Edition Section 11-5 pg 602

Angle of backfill to the horizontal $\beta := 0 \cdot deg$

Angle of internal soil friction: $\phi := 32 \cdot deg$

$$K_{p_rank} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi)^2}}$$

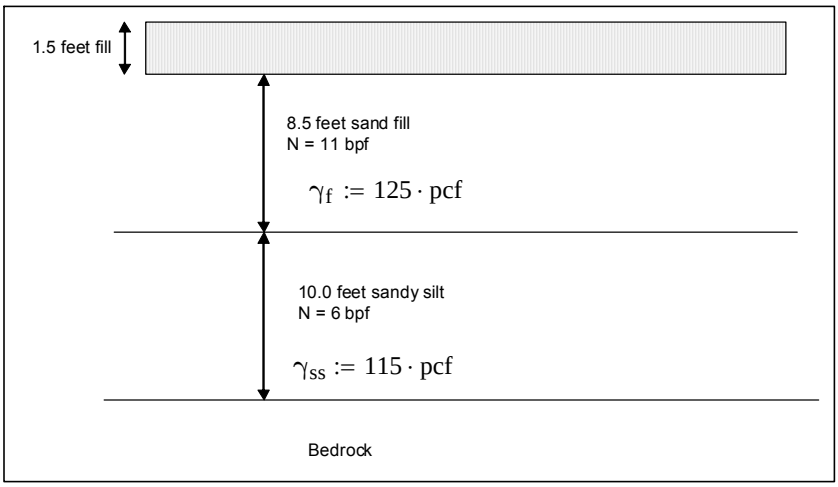
$K_{p_rank} = 3.25$

Bowles does not recommend the use of the Rankine Method for K_p when β>0.

Settlement Analysis:

Reference: FHWA Soils and Foundations Reference Manual - Volume 1 (FHWA NHI-06-088) Section 7.4.1 pg 7-16

Look at fill of 1.5 feet behind Abutment No 1:
Look at BB-WWS-101 soil profile



Water table at 8.5 ft

$\gamma_w := 62.4 \cdot pcf$

Layer 1: $H_1 := 8.5 \cdot ft$ $N_1 := 11$

Layer 2: $H_2 := 10.0 \cdot ft$ $N_2 := 6$

LOADING ON AN INFINITE STRIP
UNIFORM VERTICAL LOADING

Project Name: Canal Street Bridge
Project Number: 17085.00
Date: 12/08/10

Client: Wilton
Project Manager: NBenoit
Computed by: km

Width of strip b = 20.00(ft)
p load/unit area = 187.50(psf)

INCREMENT OF STRESSES FOR Z-DIRECTION
X = 0.00(ft)

Z (ft)	Vert. Δz (psf)
0.00	187.50
1.00	187.42
2.00	186.89
3.00	185.56
4.00	183.24
5.00	179.90
6.00	175.65
7.00	170.68
8.00	165.19
9.00	159.38
10.00	153.43
11.00	147.48
12.00	141.63
13.00	135.95
14.00	130.49
15.00	125.28
16.00	120.33
17.00	115.64
18.00	111.20

at 4.25 feet $\Delta\sigma_{z1} := 182.5 \cdot psf$

at 13.5 feet $\Delta\sigma_{z2} := 133.19 \cdot psf$

Layer 1: $H_1 := 8.5 \cdot ft$

$tsf := g \cdot \left(\frac{ton}{ft^2}\right)$

Unit weight of sand fill: $\gamma_f := 125 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$':

Calculate vertical stress: $\sigma_{1o} := \frac{H_1}{2} \cdot \gamma_f$ $\sigma_{1o} = 0.2656 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_1 = 11$

At P_o = 0.27 tsf $C_{N1} := 1.5$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60_1} := C_{N1} \cdot N_1$ $N1_{60_1} = 17$

From Figure 7-7 page 7-17 using the "clean well graded fine to coarse SAND" curve

Bearing Capacity Index: $C1 := 63$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z1} = 182.5 \cdot psf$

Layer 2: $H_2 := 10.0 \cdot ft$

Unit weight of sandy silt: $\gamma_{ss} := 115 \cdot pcf$

Determine corrected N-value normalized for overburden $N1_{60}$':

Calculate vertical stress: $\sigma_{2o} := H_1 \cdot \gamma_f + \frac{H_2}{2} \cdot (\gamma_{ss} - \gamma_w)$ $\sigma_{2o} = 0.6628 \cdot tsf$ at mid-point

Corrected SPT N_{60}-value (bpf) $N_2 = 6$

At P_o = 0.67 tsf $C_{N2} := 1.1$ From Figure 3-24 pg 3-57

Corrected N-value normalized for overburden $N1_{60}$: $N1_{60_2} := C_{N2} \cdot N_2$ $N1_{60_2} = 7$

From Figure 7-7 page 7-17 using the "well graded fine to medium silty SAND" curve

Bearing Capacity Index: $C2 := 47$

Use STRESS to determine the change in stress at the mid point of the layer under consideration (above)

$\Delta\sigma_{z2} = 133.19 \cdot psf$

Settlement at each layer:

$$\Delta H_1 := H_1 \cdot \frac{1}{C1} \cdot \log\left(\frac{\sigma_{1o} + \Delta\sigma_{z1}}{\sigma_{1o}}\right) \qquad \Delta H_1 = 0.21 \cdot in$$

$$\Delta H_2 := H_2 \cdot \frac{1}{C2} \cdot \log\left(\frac{\sigma_{2o} + \Delta\sigma_{z2}}{\sigma_{2o}}\right) \qquad \Delta H_2 = 0.11 \cdot in$$

Total settlement at Abutment No. 1= $\Delta H_{A1} := \Delta H_1 + \Delta H_2$ $\Delta H_{A1} = 0.31 \cdot in$

Frost Protection:

Method 1 - MaineDOT Design Freezing Index (DFI) Map and Depth of Frost Penetration Table are in BDG Section 5.2.1.

From the Design Freezing Index Map:
Wilton, Maine
DFI = 1800 degree-days

From the lab testing: soils are coarse grained with a water content = ~10%

From Table 5-1 MaineDOT BDG for Design Freezing Index of 1800 and wc =10%
Frost Penetration = 90.1 inches

$Frost_depth := 90.1in$ $\quad$ $Frost_depth = 7.5 \cdot ft$

Method 2 - Check Frost Depth using Modberg Software

Closest Station is Farmington

--- ModBerg Results ---

Project Location: Farmington, Maine

Air Design Freezing Index	=	2023 F-days
N-Factor	=	0.80
Surface Design Freezing Index	=	1618 F-days
Mean Annual Temperature	=	41.2 deg F
Design Length of Freezing Season	=	145 days

Layer #:Type	t	w%	d	Cf	Cu	Kf	Ku	L
1-Coarse	92.8	10.0	125.0	28	34	2.0	1.6	1,800

t = Layer thickness, in inches.
w% = Moisture content, in percentage of dry density.
d = Dry density, in lbs/cubic ft.
Cf = Heat Capacity of frozen phase, in BTU/(cubic ft degree F).
Cu = Heat Capacity of thawed phase, in BTU/(cubic ft degree F).
Kf = Thermal conductivity in frozen phase, in BTU/(ft hr degree).
Ku = Thermal conductivity in thawed phase, in BTU/(ft hr degree).
L = Latent heat of fusion, in BTU / cubic ft.

****
Total Depth of Frost Penetration = 7.74 ft = 92.8 in.
****

$Frost_depth_{modberg} := 92.8 \cdot in$

$Frost_depth_{modberg} = 7.7333\,ft$

Use Frost Depth = 7.5 feet for design

Seismic:

Wilton Canal Street Bridge 17085.00
Date and Time: 12/6/2010 3:43:28 PM

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
State - Maine
Zip Code - 04294
Zip Code Latitude = 44.617800
Zip Code Longitude = -070.236800
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.083	PGA - Site Class B
0.2	0.172	Ss - Site Class B
1.0	0.048	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
State - Maine
Zip Code - 04294
Zip Code Latitude = 44.617800
Zip Code Longitude = -070.236800
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class C - Fpga = 1.20, Fa = 1.20, Fv = 1.70
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.099	As - Site Class C
0.2	0.206	SDs - Site Class C
1.0	0.082	SD1 - Site Class C

Seismic Design Parameters for
2007 AASHTO Seismic Design Guidelines

Purpose - The ground motion parameters obtained in this analysis are for use with the design procedures described in AASHTO Guidelines for the Seismic Design of Highway Bridges (2007) The user may calculate seismic design parameters and response spectra (both for period and displacement), for Site Class A through E.

Description - This program allows the user to obtain seismic design parameters for sites in the 50 states of the United States, Puerto Rico and the U.S. Virgin Islands. In most cases the user may perform an analysis for a site by specifying location by either latitude-longitude (recommended) or zip code. However, locations in Puerto and the Virgin Islands may only be specified by latitude-longitude.

Ground motion maps are included in PDF format. These maps may be opened using a map viewer that is part of the software package.

Data - The 2007 AASHTO maps are based on 5% in 50 year probabilistic data from the U.S. Geological Survey data sets for the following regions: 48 conterminous states (2002), Alaska (2006), Hawaii (1998), Puerto Rico and the Virgin Islands (2003). These were the most recent data available at the time of preparation of the AASHTO maps. The AASHTO maps are labelled with a probability of exceedance of 7% in 75 years which is approximately equal to the 5% in 50 year data.

Disclaimer - Correct application of the data obtained from the use of this program and/or maps is the responsibility of the user. This software is not a substitute for technical knowledge of seismic design and/or analysis.

Appendix D

Special Provisions

SPECIAL PROVISION
SECTION 501
FOUNDATION PILES
(Rock Injector Pile Tip)

Subsection 501.10 Prefabricated Pile Tips of the Standard Specifications is amended as follows:

Pile tips for use on all piles shall be Rock Injector HP-80500 Pile Point, manufactured by Associated Pile and Fitting or approved equal. Material specifications, attachment of pile tips and seating of the piles shall be in accordance with Manufacturer's recommendations and in accordance with the Standard Specifications.

Payment will be made under:

Pay Item	Description	Pay Unit
501.903	Pile Tips – Rock Injector Point	Each

SPECIAL PROVISION
SECTION 610
STONE FILL, RIPRAP, STONE BLANKET,
AND STONE DITCH PROTECTION

Add the following paragraph to Section 610.02:

Materials shall meet the requirements of the following Sections of Special Provision 703:

Stone Fill	703.25
Plain and Hand Laid Riprap	703.26
Stone Blanket	703.27
Heavy Riprap	703.28
Definitions	703.32

Add the following paragraph to Section 610.032.a.

Stone fill and stone blanket shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following paragraph to Section 610.032.b:

Riprap shall be placed on the slope in a well-knit, compact and uniform layer. The surface stones shall be chinked with smaller stone from the same source.

Add the following to Section 610.032:

Section 610.032.d. The grading of riprap, stone fill, stone blanket and stone ditch protection shall be determined by the Resident by visual inspection of the load before it is dumped into place, or, if ordered by the Resident, by dumping individual loads on a flat surface and sorting and measuring the individual rocks contained in the load. A separate, reference pile of stone with the required gradation will be placed by the Contractor at a convenient location where the Resident can see and judge by eye the suitability of the rock being placed during the duration of the project. The Resident reserves the right to reject stone at the job site or stockpile, and in place. Stone rejected at the job site or in place shall be removed from the site at no additional cost to the Department.

SPECIAL PROVISION
SECTION 703
AGGREGATES

Replace subsections 703.25 through 703.28 with the following:

703.25 Stone Fill Stones for stone fill shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for stone fill shall be angular and rough. Rounded, subrounded, or long thin stones will not be allowed. Stone for stone fill may be obtained from quarries or by screening oversized rock from earth borrow pits. The maximum allowable length to thickness ratio will be 3:1. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (500 lbs) shall have a maximum dimension of approximately 36 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension of 12 inches (200 lbs).

703.26 Plain and Hand Laid Riprap Stone for riprap shall consist of hard, sound durable rock that will not disintegrate by exposure to water or weather. Stone for riprap shall be angular and rough. Rounded, subrounded or long thin stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (10 lbs) shall have an average dimension of 5 inches. The maximum stone size (200 lbs) shall have an average dimension of approximately 12 inches. Larger stones may be used if approved by the Resident. Fifty percent of the stones by volume shall have an average dimension greater than 9 inches (50 lbs).

703.27 Stone Blanket Stones for stone blanket shall consist of sound durable rock that will not disintegrate by exposure to water or weather. Stone for stone blanket shall be angular and rough. Rounded or subrounded stones will not be allowed. Stones may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (300 lbs) shall have minimum dimension of 14 inches, and the maximum stone size (3000 lbs) shall have a maximum dimension of approximately 66 inches. Fifty percent of the stones by volume shall have average dimension greater than 24 inches (1000 lbs).

703.28 Heavy Riprap Stone for heavy riprap shall consist of hard, sound, durable rock that will not disintegrate by exposure to water or weather. Stone for heavy riprap shall be angular and rough. Rounded, subrounded, or thin, flat stones will not be allowed. The maximum allowable length to width ratio will be 3:1. Stone for heavy riprap may be obtained from quarries or by screening oversized rock from earth borrow pits. The minimum stone size (500 lbs) shall have minimum dimension of 15 inches, and at least fifty percent of the stones by volume shall have an average dimension greater than 24 inches (1000 lbs).

Add the following paragraph:

703.32 Definitions (ASTM D 2488, Table 1).

Angular: Particles have sharp edges and relatively plane sides with unpolished surfaces
Subrounded: Particles have nearly plane sides but have well-rounded corners and edges
Rounded: Particles have smoothly curved sides and no edges